UNIVERSITY OF NAIROBI

Characterisation of Asphalt Mixtures for Permanent Deformation

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Declaration

This thesis is my original work and has not been presented for a degree in any other University

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This thesis has been submitted for examination with our approval as University supervisors

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Prof. Sixtus Kinyua Mwea

Abstract

The study used triaxial compression test to characterize asphalt mixtures for permanent deformation. Various factors both structural and environmental that affect the permanent deformation of asphalt mixtures were investigated with an aim of understanding how they contribute to the permanent deformation of asphalt mixtures.

The main aim of the study was to characterize asphalt mixtures for permanent deformation which was achieved by setting out six objectives namely; to investigate the effects of aggregates gradation, loading temperature, bitumen content, confining stress and bitumen consistency on permanent deformation of asphalt mixtures and finally to identify suitable models that best describe permanent deformation of asphalt mixtures.

Asphalt mixtures made of well graded aggregate gradation were found to have high resistance to permanent deformation as compared to asphalt mixtures made of the gap-graded aggregate gradation. Permanent deformation of asphalt mixtures was found to increase with increase in loading temperature. Asphalt mixtures made of highly consistent bitumens (60/70 penetration grade bitumen) were found to have high resistance to permanent deformation as compared to the asphalt mixtures made of low consistent bitumens (180/200). High bitumen content was found to increase the permanent deformation of asphalt mixtures. High confining stresses were found to increase the resistance to permanent deformation of asphalt mixtures. Gap – graded asphalt mixtures made were found to be more susceptible to permanent deformation at high loading temperatures while their continuously graded counterparts were found to be less susceptible to permanent deformation at high temperatures. Continuously graded asphalt mixtures with low contents of highly consistent bitumen were found to be more suitable in resisting permanent deformation at high temperatures and high confining stresses.

Logarithmic linear regression and power law equations were identified as the most suitable models to describe the test results for permanent deformation. Tangent stiffness (T_S) was related with the loading temperature (T) using a natural logarithmic linear regression relationship of the form $\ln T_S = C_1 + k_1 \ln T$ where C_1 and k_1 are material constants. The maximum stress (f_{ca}) was

related with the confining stress (σ_3) by a power law of the form $f_{ca} = a\sigma_3^b$ where *a* and *b* are model parameters. All the correlations were of high coefficients of determination (R^2) ranging between 0.96 and 0.98.

Dedication

This thesis is dedicated to all the Highway Engineers in Kenya for their unseen, unheard, unacknowledged hard work and dedication to the public, and to my wife Anne and our daughter Aspasia Wanjiku without whose support this thesis would not have been written.

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I would like to take this opportunity to convey my heartfelt gratitude and acknowledge the help, support and guidance of many individuals and Institutions. Foremost I would like to thank my supervisor Prof.F.J.Gichaga who regardless of his busy office of Professor, Consultant engineer and a university Chancellor found time to lead me intellectually through the research program right from the beginning to the end.

My thanks also go to the University of Nairobi as an Institution through whose scholarship I was able to conduct my master's study. I thank the entire civil engineering department administration and technical staff especially Prof S.K.Mwea, the Chairman and those ones in the Highways and Soil laboratories for their contribution in making my studies a success.

No matter where I go or what I do, my family has always been there for me, prayed for me and have been my source of inspiration. Thank you for your patience, support, love, care, sacrifices and everything else. I am what I am today because of you. I thank my friends and for all your prayers and best wishes.

Last but not least, I thank God the Almighty for all the blessings.

Table of Contents

List of Figures

List of Tables

List of Plates

List of Abbreviations

ACV	Aggregate Crushing Value
ADO	Actual Design Optimum Bitumen Content
AIV	Aggregate Impact Value
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ASTM	American Society for Testing and Materials
BS	British Standard
CDM	Compacted Density Mix
CBR	California Bearing Ratio
CEBTP	Centre for Experimental Research and Studies of Building and Construction
DAC	Dense Asphalt Concrete
DBM	Dense Bitumen Macadam
FADO	First approximated Design Optimum Bitumen Content
HMA	Hot Mix Asphalt
LAA	Los Angeles Abrasion
MDD	Maximum Dry Density
MoR	Ministry of Roads
Mpa	Mega Pascal
OMC	Optimum Moisture Content
PAC	Porous Asphalt Concrete
PI	Penetration Index
SG	Specific Gravity
SGM	Specific Gravity of the Mix
SMA	Stone Mastic Asphalt Concrete
VFB	Voids Filled With Bitumen
VIM	Voids in the Compacted Mix
VMA	Voids in the Mineral Aggregates
VMAC	Critical Voids in the Mineral Aggregates
VRSPTA	Vehicle Road Surface Pressure Transducer Array

List of Symbols

γ = shear strain

ε = Strain

f_{ca} = Maximum stress.

σ_3 = All round pressure (confining pressure)

a, b = Model parameters.

C = Cohesion

C_1, k_1 = Materials constant

C_u = coefficient of uniformity

C_v = volume concentration of aggregates

C_z = coefficient of curvature

e = vertical permanent deformation

E = Young's Modulus

I_1= the first stress invariant

J_2 = the second deviatoric stress invariant

J_3 = the third deviatoric stress invariant

N = Load application number

ø = Angle of internal friction

P = Isotropic stress

S_b = stiffness modulus of bitumen

S_i = i^{th} principal deviator stress

S_m = stiffness modulus of bituminous mixtures

T = loading temperature

t = Time

T_S = tangent stiffness

V = Volume of specimen

V_b = Volume percentage filled by bitumen in the bituminous mixture

V_g = Volume percentage filled by aggregates in the bituminous mixture

V_p = Compression wave velocity evaluated from seismic refraction testing

V_s = Shear velocity obtained from vibration testing

μ = Poisson's ratio

σ = Normal stress

σ_1 = Axial stress with its value at failure called shear stress at failure or maximum stress

τ = Shear strength

Chapter one

Introduction

1.0 Background

A significant budget is allocated to the road sector in Kenya each year. This is just an indication of how important the road sector is in Kenya. Good roads are associated with increased level of developments. This is because people and goods can easily flow which attracts investment into a given area. As a result it is found that the value of land and the properties in areas well covered by roads, increases. It therefore becomes very important to use more comprehensive and accurate design methods leading to strong pavements which are able to achieve their design life. Strong pavements will lead to cost reduction in terms of road maintenance. Transportation of goods, people and services from one point to another plays an important role in our daily lives. Road transportation creates a vital link in transportation networks. The ability of roads or pavements to deliver goods, people and services within acceptable costs in terms of comfort time and safety is governed by the pavement life. Permanent deformation or rutting is one of the important failure modes in asphalt pavements according to a University of Michigan report (Barksdale, 1972) that affects the pavement life. Permanent deformation in asphalt mixtures can be defined as the unrecoverable cumulative deformation that occurs mainly at high temperatures in the wheel paths as a result of repeated traffic loading. The deformation results in depression on the pavement surface along the wheel tracks relative to other points on the surface (Rabbira, 2014). The depressions are as a result of downwards and lateral movements of asphalt mixture. The downward movement is mainly due to compaction while the lateral movement occurs as a result of shear failure. Rutting as a form of permanent deformation is different from slip in that slip is a form of pavement permanent deformation failure which occurs as a result of slope instability in soil embankments.

Most of the design methods used in Kenya for flexible pavements were borrowed from Europe. Direct use of these design methods in tropical countries has in several cases led to road pavement failures sooner than expected thereby leading to unplanned expenditure in the exercise of rehabilitating failed roads (Gichaga, 1982). As long as conditions under which the design methods such as empirical methods were developed prevail, pavements would perform satisfactorily

Gichaga (1982) carried out a field study to determine the various types of distress features of flexible pavements in Kenya and found them to be cracks, potholes, severe deformations (heave/rutting), shear failure, edge failure, surface ravelling and fretting, breaking up of patched areas, poor trench reinstatement, poor verge maintenance and poor drainage maintenance (Gichaga,1982). In one way or another distress features, if not looked into, will lead to permanent deformation especially rutting and heaving. The study concluded that pavements give higher deflections with increase in repetitions of traffic loads and that they develop strength with age. Pavements made up of cement stabilized murram (lateritic gravel) base showed lower elastic deflection than that made of crushed stone bases. Pavements made up of cement stabilized murram base showed less rutting than that made up of crushed stone base.

Pavements made up of crushed stone base showed high rate of cracking than that made up of cement stabilized murram base. Higher elastic deflections are obtained during the months of high rainfall and high air temperatures. For a pavement approaching failure, elastic deflections tend to increase with increase in cracking. Road pavements with surface dressing as a form of surfacing have continued to perform well for periods in excess of fifteen years despite heavy traffic loading in the pavements studied.

Gichaga (1982) classified pavement design methods under two basic categories namely empirical and theoretically based methods. Many semi-empirical and empirical design procedures include modifications of the CBR design method which was originally developed by the California Division of Highways e.g. Road Note 31, Road Note 29, AASHTO interim guide, Shell Pavement Design method and CEBTP Pavement Design methods (Gichaga, 1982).

In order to offer an acceptable pavement life, several factors play a great role on the pavement deformation resistance of asphalt mixtures. These factors can either be economical or environmental. Great demands are placed on the permanent deformation resistance by the increase in axle loads as a result of economic and environmental factors. Economical factors such as tendency to transport goods in large units and reduction of rolling resistance leads to the placement of heavier axle loads on fewer tyres. This is so especially where the government regulations require that the maximum axles for trucks be limited to a certain number. When the government agents like the traffic police who are responsible for the implementation of the legislation are not keen on controlling the maximum load carried by the trucks, a scenario results where there are heavier axle loads on fewer tyres which have an effect of increasing the contact stresses leading into permanent deformation of the asphalt pavements. Environmental pressure to reduce the number of scrap tyres also means more pressure to reduce the number of tyres which leads to a scenario discussed in the latter sentence.

The above discussion emphasizes the need for materials with improved characteristics to cater for great demands that are placed on the permanent deformation resistance of asphalt mixtures. Unfortunately the knowledge of permanent deformation is mainly empirical and is suitable for materials and conditions under which the knowledge was attained. Empirical knowledge does not provide insight into the contribution of different materials characteristics that are otherwise known to affect the permanent deformation behaviour.

In addition most of the studies carried out on permanent deformation consider asphalt mixture as a homogenous material. This has also led to permanent deformation models that do not provide insight in the contribution of different material characteristics that are otherwise known to affect the permanent deformation behaviour of asphalt mixtures. It is the aim of this research to address the contribution of different material characteristics towards the resistance into permanent deformation.

1.1 Definition of Asphalt Mixtures

Asphalt concrete is a group of bitumen bound materials used as pavement surfacing. They normally consist of a mixture of coarse aggregate, fine aggregate and filler bound with straight – run bitumen (Kenyan Road Design Manual, 1987). The proportions and the grading of the coarse aggregate are normally varied to produce different types of mix with differing properties. Bituminous binders are petroleum derived adhesives used to stick chippings together in a road surface, in surface dressings or to bind together a layer of surfacing or base material. Straight – run bitumen is defined as bitumen whose viscosity or composition has not been adjusted by blending with solvents or any other substance. On the other hand, cut–back bitumen is bitumen whose viscosity has been reduced by addition of volatile solvent. Asphalt concrete can either be sand asphalt or gap – graded asphalt where sand asphalt refers to surfacing material consisting of hot - mixed, hot-laid, plant mixture of natural sand and, in some cases mineral filler and crushed fine aggregate bound with straight – run bitumen. Gap – graded asphalt is a hot laid, plant mixture of gap-graded aggregate, filler and straight – run bitumen, used for pavement surfacing.

There are two main types of bituminous mixes namely; Gap graded bituminous mixes and continuously graded bituminous mixes. Gap graded bituminous mixes are also known as hot rolled asphalts and they normally consist of bitumen plus three separate and distinct sizes of aggregates namely stone , sand and the filler. Continuously graded bituminous mixes are also called asphalt concrete. They are normally made with an aim of achieving a blend of aggregates so proportioned that they will compact to give a dense interlocked structure when bitumen is added to the mix to become almost impervious with viscous and elastic properties.

1.2 Problem Statement

In Kenya there has been high incidence of asphalt pavement failure immediately after construction and commissioning. This fact supports the suspicion that, there are changes in the stress-strain behaviour of bituminous mixtures when exposed to climatic factors and traffic loading. These changes in the stress – strain behaviour lead to changes in strength properties of asphalt mixtures and thus cause them to perform poorly as pavement materials. The changes in stress-strain behaviour are supposed to have been considered during the design stage so as to help the pavement designer come up with asphalt mixtures which are capable of resisting permanent deformation. The resultant poor performance of the bituminous binders and bituminous mixtures can be summed up in terms of the pavement distresses that result.

Permanent deformation in the form of rutting is one of the most important distress (failure) mechanisms in asphalt pavements. With increase in truck tire pressures in recent years, rutting has become the dominant mode of flexible pavement failure in Kenya. Pavement rutting, which results in a distorted pavement surface, is primarily caused by the accumulation of permanent deformation in all or a portion of the layers in the pavement structure. Longitudinal variability in the magnitude of rutting causes roughness. Water may become trapped in ruts resulting in reduced skid resistance, increased potential for hydroplaning and spray that reduces visibility. Progression of rutting can lead to cracking and eventually to complete disintegration or failure. Rutting accounts for a significant portion of maintenance and associated costs in both main highways and secondary roads.

The economics of truck transportation has caused the average gross weight of trucks to increase so that a majority of trucks are operating close to or higher than the legal axle load limits. In countries where enforcement of the legal axle load limits is relaxed or non-existent (typical of developing countries such as Kenya), trucks operate at axle loads which by far exceed the legal axle load limit. As axle loads have increased, the use of higher tire pressures has become more popular in the transport industry. Higher tire pressures reduce the contact area between the tire and the pavement, resulting in high stress which contributes to greater deformation in flexible pavements, manifested as severe wheel track rutting especially on upgrade sections of a pavement. As a consequence of the increased tire pressure and axle load, the surfacing asphalt layer is subjected to increased stresses, which result in permanent (irrecoverable) deformations. The permanent deformation accumulates with increasing number of load applications. The permanent deformation in the surfacing layer thus accounts for a major portion of rutting on flexible pavements subjected to heavy axle loads and high tire pressures.

Although the rutting observed on flexible pavements can be the total sum of accumulated permanent deformations in one or more layers of the pavement structure, the accumulation of permanent deformation in the asphalt surfacing layer is now considered to be the major cause of rutting. To minimize this form of rutting, it is necessary to pay extra attention to material selection and mixture design. To be able to design a mixture that has adequate resistance to rutting, knowledge of the effect of mixture composition and properties of the component materials is of paramount importance. Furthermore, the questions of how to measure permanent deformation of asphalt mixtures, what parameters to use as a measure of resistance, and how to model the permanent deformation need to be addressed.

Many previous research works on the characterization of asphalt mixtures are based on uniaxial laboratory test methods. The uniaxial methods are not able to simulate the actual loading conditions of real pavements because they only consider vertical stresses while in reality it is known that lateral loads are present on pavements under traffic loading. It therefore goes without saying that the data obtained using these uniaxial methods do not reflect the actual effects of traffic loads on asphalt pavements. There is a need therefore to come up with a lab testing method capable of simulating the actual loading conditions of asphalt pavements under the influence of traffic loading and climatic conditions. Such a method will help in developing a realistic characterization of asphalt mixtures for permanent deformation.

An attempt is made in this study to tackle the issues raised in the preceding paragraphs. Based on laboratory tests that are judged to be simulative of field loading conditions, the study attempts to provide more knowledge on the effect of volumetric composition (gradation and bitumen content), bitumen consistency ,loading, and temperature conditions on permanent deformation response of asphalt concrete mixtures. In particular substantial effort is made to evaluate the effects of volumetric composition and loading conditions on the permanent deformation of asphalt mixtures. Modelling the permanent deformation behaviour of asphalt concrete mixtures forms the other major part of this study. Modelling is done by use of constitutive equations which enable the researcher to relate the various measured parameters. In return predictive equations are developed.

Asphalt mixtures are made of bitumen binders and aggregate skeletons. Bitumen binders are visco – elastic and their response to loading is temperature dependent. Generally, bituminous mixtures experience decrease in strength during hot seasons especially those found in tropical climates. In this state of decreased strength the materials undergo distortional distress. This means that higher stresses are transferred into the subgrade. The reverse happens during the cold season where the asphalt pavements become stiffer and are capable of carrying higher wheel loads. The stiffer pavements are however more brittle and are likely to crack and eventually disintegrate under traffic loading.

1.3 Aim of the Research

The main aim of this research was to characterize asphalt mixtures and their influence on permanent deformation. By characterizing asphalt mixtures for permanent deformation, the researcher intended to provide more insight into the effects of the different parameters both environmental and structural in asphalt mixtures towards the resistance to pavement's permanent deformation. In order to meet this aim, it was necessary to investigate different asphalt mixtures design under different conditions such as the loading temperatures, bitumen consistency and content, confining stress as well as aggregate gradation. The specific objectives of this study were;

i. To investigate the effects of gradation on permanent deformation of asphalt mixtures.
ii. To investigate the effects of bitumen consistency on permanent deformation of asphalt mixtures.
iii. To investigate the effects of bitumen content on permanent deformation of asphalt mixtures.
iv. To investigate the effects of loading temperature on permanent deformation of asphalt mixtures.
v. To investigate the effects of confinement on permanent deformation of asphalt mixtures.
vi. To identify models that best describe permanent deformation of asphalt mixtures

Chapter two

Literature Review

2.1 Introduction

This chapter gives a brief description of various issues related to permanent deformation. Several factors that influence permanent deformation behaviour of asphalt mixtures are presented. A discussion on the influence of the different components in an asphalt mixture and an outline on the modeling of permanent deformation are presented.

2.2 Factors Affecting Permanent Deformation

Generally, it is accepted that permanent deformation that is observed at the pavement surface in the wheel tracks is influenced by the stress conditions and properties of pavement layers. The stress conditions and properties of pavement layers are influenced by the contact stress distributions as well as the stiffness and thickness of the various pavement layers. The stiffness and resistance to permanent deformation of asphalt mixtures strongly depends on the mixture composition, degree of compaction, rate of loading and temperature. The properties of asphalt mixtures are found to be well established as highly dependent on microstructure including air voids, aggregates and mastic (Norhidayah et al, 2012).

2.2.1 Contact Stress Distribution

The wheel load will normally generate non-uniform vertical, longitudinal and lateral contact stresses. The contact stresses are a function of tyre pressure, wheel load, type of tyre and pavement surface texture depth. Himeno et al (1997) studied the effects of speed, vehicle load and tyre inflation pressure on the vertical contact stress distribution. Firstly they concluded that the distribution of vertical contact stress was complex. Secondly, they found that the relationship between the average vertical contact stress and the wheel load is dependent on tyre inflation pressure and vehicle speed. Knowledge of average contact stresses is not however sufficient to analyse permanent deformation in detail and more detailed information on the distribution of the contact stresses is needed. This can be retrieved from the work done by De Beer et al (1997). In their research, De Beer et al (1997) carefully measured the contact stress distribution of a free rolling wheel at a slow speed using a sophisticated measuring system known as Vehicle Road Surface Pressure Transducer Array (VRSPTA). In this study it was found out that contact stresses consist of three dimensions; vertical, lateral, and longitudinal contact stresses and that the contact stresses depend on the tyre inflation pressure, tyre type and the wheel load.

Groenendijk et al (1997) investigated the effect of tyre inflation pressure, tyre load and speed, on the contact stress using two different types of tyres. They found out that the vertical, lateral and longitudinal contact stresses are influenced by tyre pressure, tyre load and speed. However, the speed could only be varied between 0.32 to 4m/s. De beer et al and Groenendijk showed that peak contact stresses that can result in significant rutting can be avoided if wheel load and tyre pressure are balanced with respect to each other. Woodside et al (1992) studied the effect of the surface texture depth on contact stresses. The surface texture depth in this study was simulated by varying the height of transducer protruding from a simulated road surface. The findings from this study indicated that a surface chipping of 1mm macro texture endured a contact force of twice that induced on a chipping at zero texture depth.

Many pavement analysis methods assume that a uniform vertical contact stress is sufficient to characterize the contact stress arising from the wheel load. Using a multi-layer linear elastic approach, Verstraeten (1967) demonstrated that serious errors can be made in the magnitudes and distributions of the stresses in the upper pavement layers (depth to contact radius ratio $\leq$ 0.5) if the shear stresses on the surface of the pavement are ignored. In addition he illustrated that the failure conditions cannot be simply characterized by only one point in a three dimensional system but a particular surface at which the stress combinations became critical to the strength of the material.

Heavy slow moving vehicles are known to cause considerable rutting on upgrade sections of asphalt pavements as demonstrated by a report compiled by Harun and Jones (1992). At first sight, it may appear as if this considerable rutting is caused by the slow speed of vehicles. However, this considerable rutting can be associated with the increase in longitudinal and lateral contact stresses. The increase in these stresses results from the increase in the component of the wheel load acting on the inclined surface.

2.2.2 Pavement Structure

A typical asphalt pavement structure consists of a top layer, base, subbase and subgrade, as shown in Figure 2.1 (Huang, 1993). The distribution of stresses in the pavement depends on the stiffness, poisson's ratio and thickness of pavement layers. Just like granular materials, the resistance to permanent deformation of asphalt mixtures is to a large extent dependent on the magnitude of confining stresses that result from the wheel load. An increase in confinement enhances the resistance to permanent deformation. By carefully stacking the pavement layers, significant confinement levels can be generated leading to enhanced resistance to permanent deformation. Such a stacking will mostly involve a layer with a high modulus below the asphalt top layers.

Top layer

Base

Subbase

Subgrade

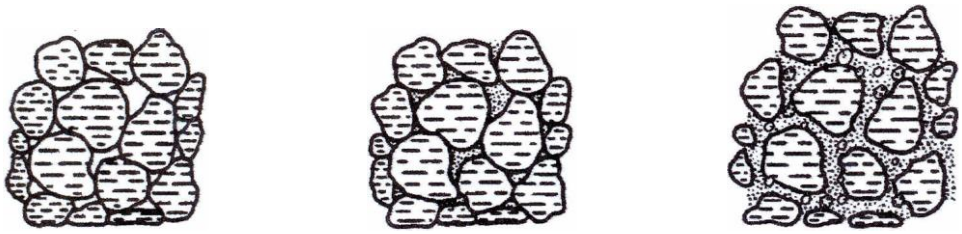
Figure 2.1; Pavement Structure

2.3 General Concept

Asphalt mixtures share many similarities with soil mixtures. Figures of physical states of soil mixtures and X-ray tomography images of porous asphalt concrete (PAC), stone mastic asphalt concrete (SMA) and dense asphalt concrete (DAC) show many similarities as illustrated in Figures 2.2(a) and 2.2(b).

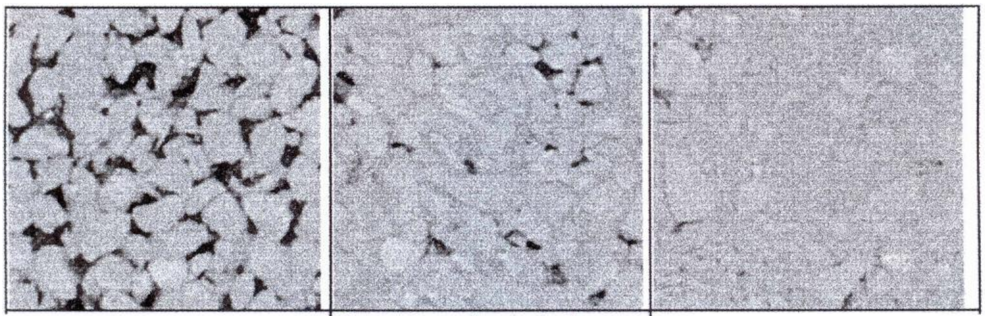
At first sight, the soil mixture gives impression that the stone skeleton held together by the mortar plays an important role in offering resistance to permanent deformation. Initially the friction forces generated in the contact points carry the loads applied to the skeleton and skeleton will only show a limited amount of deformation. If however the load becomes too high, deformations will occur and because the skeleton is so well compacted it will not show a decrease in volume but an increase as a result of dilation. When dilation occurs, the mortar is subjected to tension hence underlining the importance of adhesive characteristics of the mortar to the aggregate and the tensile characteristics of the mortar as shown in Figure 2.2(a-i) which resembles porous asphalt concrete.

The soil mixture as shown in Figure 2.2(a-ii) bears good resemblance to the SMA. In this case, the resistance to permanent deformation relies on both the skeleton and the mortar. If the void content is low, pore pressures as observed in saturated soils might develop resulting in instability of the mixture.



(i) (ii) (iii)

Figure 2.2(a); Physical states of soil mixtures,(Yoder 1959)



Porous asphalt concrete (PAC) | Stone mastic asphalt concrete (SMA) | Dense asphalt concrete (DAC)

Figure 2.2(b); X-*ray tomography images of porous asphalt concrete (PAC), stone mastic asphalt concrete (SMA) and dense asphalt concrete (DAC), (Muraya 2007)*

Furthermore, the tensile characteristics of the mortar and adhesive characteristics of the mortar aggregate interface become important if dilation occurs. The soil mixture shown in Figure 2.2(a-iii) is compared to the DAC. In such a mixture, the large aggregate stones do not really form a skeleton but more or less float in the mortar. The large aggregate particles do not play a role in providing resistance to permanent deformation. This resistance must be provided by the mortar thus emphasizing the importance of the visco-elastic properties in such a mixture. The type of asphalt, type of filler and the amount of asphalt and filler mainly determine these properties. If the void content is too low, pore pressure effects as observed in saturated soils may occur resulting in instability of the mixture.

The preceding discussion shows that asphalt mixtures share many similarities with soil mixtures and it also demonstrates that the skeleton and the mastic play specific roles. The discussion also underscores the importance of adhesive and tensile characteristics of the mortar. Therefore, it is important to understand and characterize the contribution of the different components of asphalt mixture in development of its resistance to permanent deformation.

2.3.1 Mixture Composition

The influence of mixture composition can be seen in relation to the aggregate skeleton, the mortar or mastic and the voids in asphalt mixture. The mortar in this research can be defined as a mixture of bitumen and any other aggregates below a certain aggregate size and the mastic is a mixture of bitumen and filler. The mineralogy and amount of filler influences the behaviour of mastic and the mortar and consequently the behaviour of asphalt mixture. In addition the influence of mixture characteristics depends on the gradation and the surface texture of the aggregates.

Gradation is the particle size distribution in an aggregate mixture and is determined in terms of the percentage passing or retained on each of the sieves. In order to ensure acceptable pavement performance, gradations are normally specified. For example Superpave specifies gradations within designated control points (Roberts et al 1996). Gradations passing through restricted zones are thought to have low resistance to permanent deformation but this has been disapproved in some studies as presented later.

Gradation influences the permanent deformation in asphalt mixtures. Asphalt mixtures composed of different gradations but of similar mineralogical composition exhibit significantly different permanent deformation behaviour. This was shown in a study conducted by Kandhal and Mallick (1999) in which the effect of gradation on the permanent deformation behaviour of Superpave dense asphalt mixtures was evaluated. In this study, different gradations with similar mineralogical composition and similar gradations with different mineralogical composition were evaluated. The asphalt mixture was composed of granite, limestone and gravel gradations passing above, through and below restricted zone. The results of this study showed significant gradation effects on the permanent deformation behaviour of asphalt. For granite and limestone mixtures, the gradation passing below the restricted zone exhibited the highest amount of rutting; gradations passing through showed the lowest amount of rutting and the gradations passing above showed intermediate amount of rutting. For gravel mixtures, the gradation passing below the restricted zone exhibited the lowest amount of rutting while the gradations above showed the highest amount of rutting and gradations passing through showed an intermediate amount of rutting, (Kandhal and Mallick ,1999)

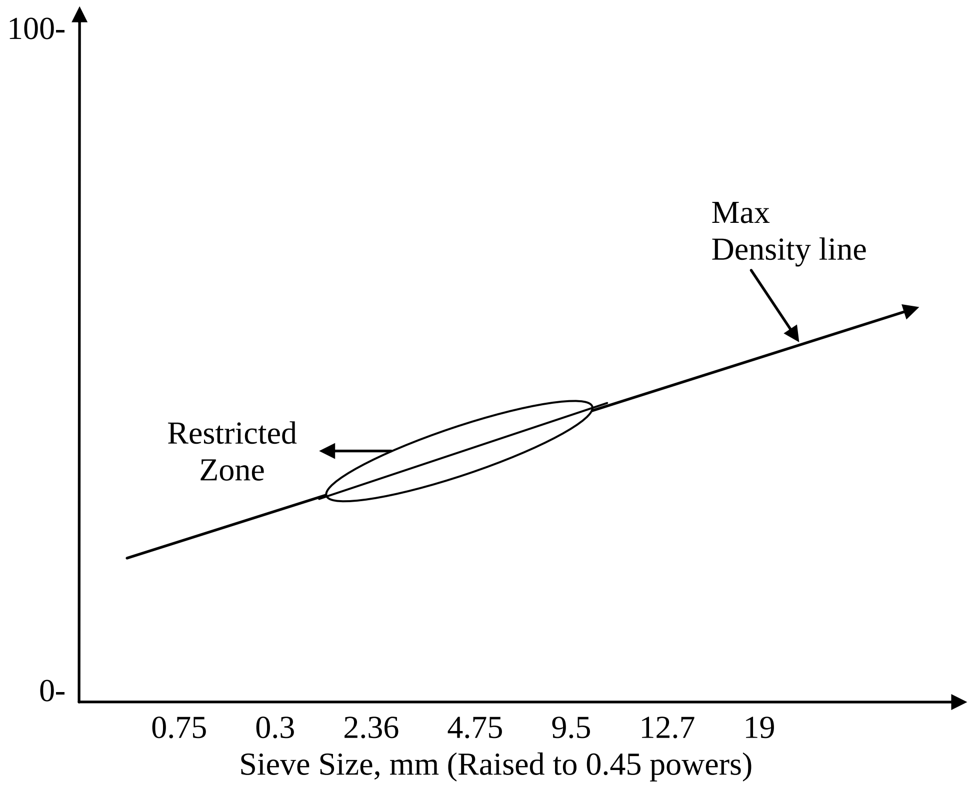


Figure 2.3; Superpave gradation for 12.7mm nominal max size (Kett 1998)

The perception that gradations passing through the restricted zone offer tenderness and hence low resistance to permanent deformation was also disapproved in a study conducted by Venn Van de et al (1997). The study considered dense asphalt mixtures made from different gradations but with similar mineralogical composition. Part of the study involved an evaluation of rutting resistance of the asphalt mixtures. The evaluation suggested that the restricted zone does not necessarily influence the rutting resistance of asphalt mixtures.

The type of aggregate surface texture depends on hardness, grain size, pore characteristics of the parent rock as well as the extent to which forces acting on the particle surface have smoothened or roughened the surface (Su, 1996). The type of aggregate surface texture is categorized by the shape parameters such as angular, rounded, smooth or rough. The aggregate surface texture affects the aggregate interlock. Durable angular aggregates with rough surface texture are normally considered to offer good aggregate interlock.

The type of aggregate surface texture influences permanent deformation of aggregate mixtures. This was shown by Kalcheff and Tunnicliff (1982) who investigated the effects of crushed stone aggregate size and shape in asphalt mixture properties. They concluded that mixtures containing crushed coarse and fine aggregates with or without high proportions of mineral filler should be more resistant to permanent deformation resulting from repeated traffic loadings. They also found that these asphalt mixtures were much less susceptible to the effects of temperature and high initial void content than comparable mixtures containing natural sand. However, this is countered by Celard (1977) who showed that mixture containing a large fraction of rounded aggregates could exhibit better dynamic creep behaviour than the same composition made up of crushed aggregates only.

A similar finding was made by Coree and Hislop (2001) who conducted their investigation at constant compaction but using different gradations of continuously graded mixtures. They defined the critical voids in the mineral aggregates (VMAC) as the VMA at which a mixture transits from sound to unsound permanent deformation behaviour i.e. the VMA below which excessive permanent deformation initiates. They found this critical VMA to be directly related to the proportion of crushed coarse and fine aggregates and to the stiffness modulus.

In the composition of asphalt mixtures, fillers are used to meet specifications for the aggregate gradations, to increase stability and to improve the bond between the Mortar/Mastic and the aggregates. Anderson et al (1982) reported that part of the filler is embedded in the bitumen while the other part fills the voids in the aggregate mixture. The part that fills the aggregate voids provides contact points between large aggregate particles. The embedded part of the filler may act as a bitumen extender or serve to stiffen the bitumen. He also reported that the rheological behaviour of bitumen was influenced by the size of filler particles. The type of bitumen and mineralogy of the filler had a significant effect on the rheological behaviour of the mastic. Stiffening effect of fine mineral powders on filler/bitumen mixtures were relatively small at short loading times or low temperatures but large at higher temperatures and long loading times. The same review also reported that, in research carried out by Craus and Ishai (1978), it was concluded that strength of the filler – bitumen bond increased with increase in absorption intensity, geometric irregularities, (shape, angularity and surface texture) and the selective adsorption potential of fillers.

Bolk et al (1982) conducted a study on the effect of filler on the mechanical properties of dense asphalt concrete. The study considered fillers based on limestone powder and fillers based on fly ash. Results from static creep tests performed at 40°C and at stress level of 10^5 N/m^2 for a period of one hour showed that the creep stiffness was affected by the bitumen and filler content as shown in Figure 2.4.

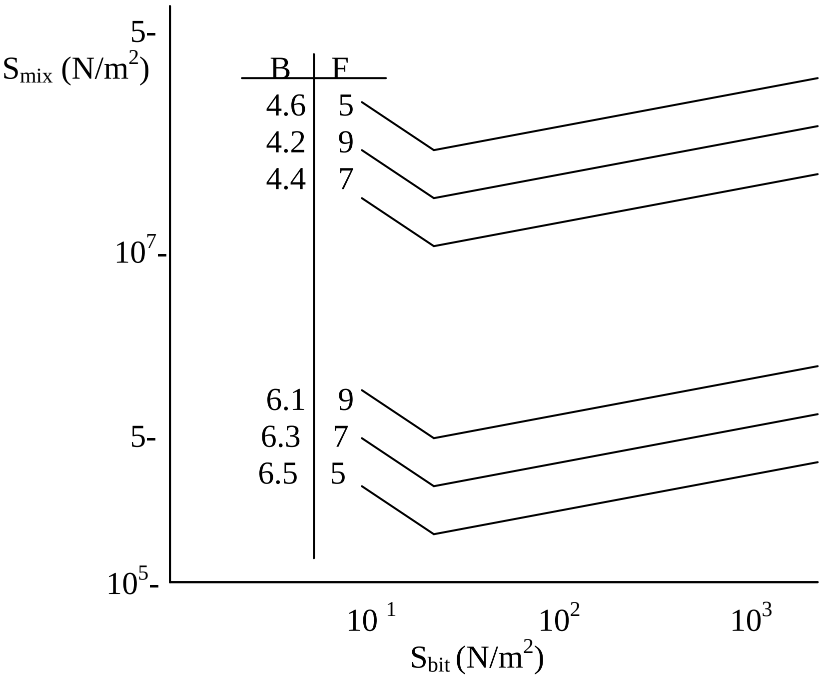


Figure 2.4; Effect of Type of Filler, Filler Content and Bitumen Content on Creep Stiffness (Bolk et al 1982).

Note; *B=Percentage by mass of the bitumen in mixture on 100% mineral aggregate.*

F=Percentage by mass of filler in the mixture.

Since the mortar/mastic is composed of bitumen and aggregate particles, change in bitumen properties influences the behaviour of the mortar/mastic. Studies performed by shell researchers (Heukelom and Wijga 1973) suggest that the stiffness of the mastic and asphalt concrete is influenced by stiffness of the bitumen and content of the filler and bitumen. As evident from Figure 2.5, increase in stiffness of bitumen increases the stiffness of mastic and the asphalt concrete. A similar trend in the stiffness of mastic is observed when content of the filler is increased. Apart from increasing the stiffness of mastic and the asphalt concrete, increase in bitumen stiffness particularly at high temperatures has also been shown to decrease the amount of permanent deformation in asphalt mixtures. A research performed by Ghile (2006) showed that modification of bitumen can enhance the bitumen stiffness and rutting behaviour of asphalt mixture. The study conducted by Ghile involved dense asphalt mixtures composed of unmodified and modified bitumen. The bitumen was modified by a special type of nanoclay. The study found that the modified bitumen not only improved the stiffness of bitumen at high temperatures but also enhanced the resistance to permanent deformation of the dense mixtures in unconfined repeated load compression tests.

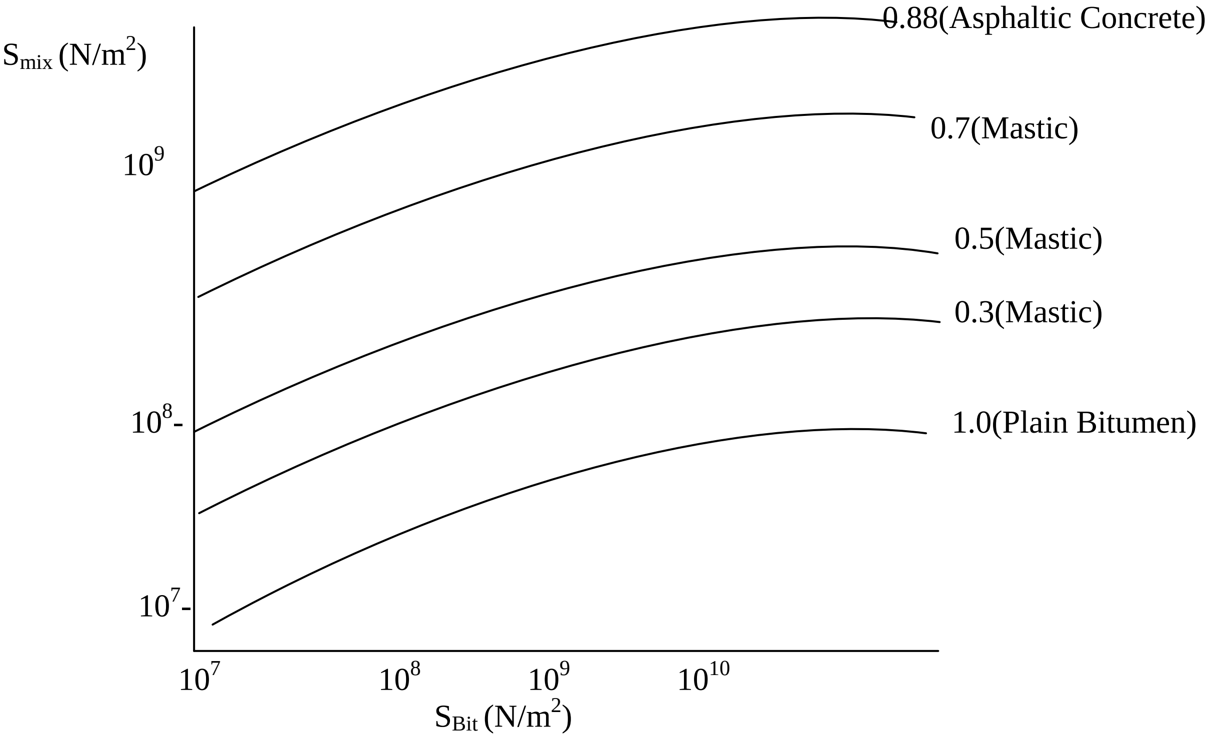


Figure 2.5; Influence of Bitumen stiffness on Mastic and Asphaltic Concrete (Heukelon and Wijga) - 1973

The amount of viscous deformation in bitumen depends on the stiffness and Penetration Index of bitumen as evident from shell bitumen testing (Heukelom and Wijga 1973). Increase of both the stiffness and the Penetration Index decreases the amount of viscous deformation in the bitumen relative to the delayed elastic deformation thereby leading to increased resistance to permanent deformation.

2.3.2 Voids in Asphalt Mixture

With regard to asphalt mixture, air voids and the voids in the aggregate skeleton filled with bitumen play a significant role in the resistance to permanent deformation. Air voids is found to be significantly affected by compaction level (Soon-Jae Lee et al, 2009). Some asphalt mixture design specifications prescribe air voids and voids filled with bitumen as part of the acceptance criteria for asphalt mixtures. For example, the Dutch mix design specifications (Crow 2000) prescribe a maximum amount of air voids content of 4 – 6% and a maximum voids filled with bitumen of 87 – 80%. The purpose of this provision is to prevent overfilling that may occur as a result of extension of volume of bitumen at high temperatures. The amount of voids filled with bitumen has been shown to influence the stiffness of asphalt mixture. During the study performed by Bolk et al (1982), the creep stiffness was found to decrease with increase in voids filled with bitumen as illustrated in Figure 2.6.

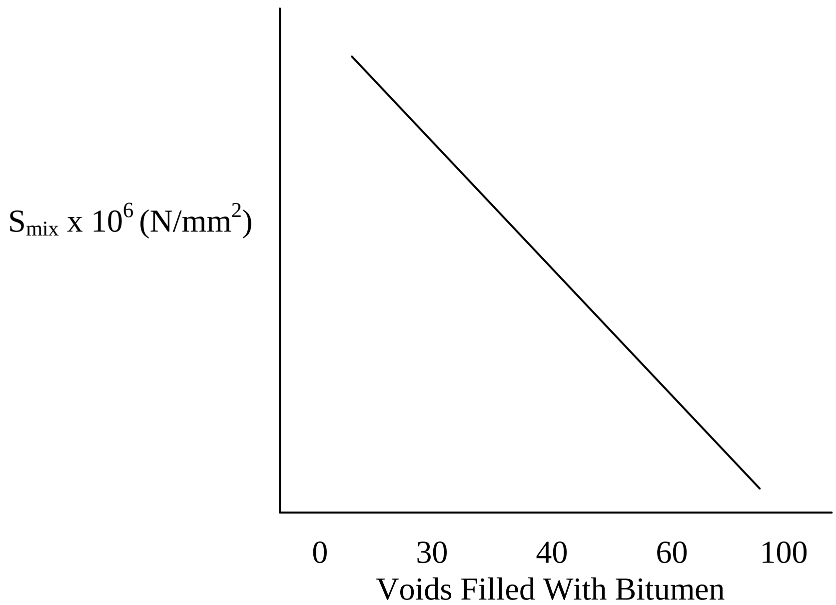


Figure 2.6; Effects of Voids Filled with Bitumen on Static Creep Stiffness (Bolk et al 1982)

The content of air voids in asphalt mixtures also affects the permanent deformation behaviour. In their literature survey on types of density specifications, Roberts et al (1996) concluded that too low air voids contents (< 3%) could lead to rutting and shoving in dense graded asphalt mixtures. May and Witczack (1992) found that rapid or plastic flow dominates the permanent deformation behaviour of HMA mixtures with less than 3% air voids. In their study, Soon-Jae Lee et al (2009) found that too much avoids will have a detrimental effect on rutting resistance.

2.3.3 Compaction

The purpose of compaction in asphalt mixtures is to stabilize and enhance the mechanical properties of asphalt mixtures. Linden and Van dertteide (1987) investigated the influence of compaction in asphalt mixtures. They concluded that the degree of compaction was a dominant quality parameter in asphalt mixtures especially when the mixture is critically designed with low bitumen content to deliver high resistance to permanent deformation in mixtures such as dense asphalt concrete. The level of field compaction is also affected by the method of compaction and compacting temperature. Soon-Jae Lee et al, (2009) found that compaction temperatures significantly affect volumetric properties of asphalt mixtures. In the Marshall Mixture design, the compactive effort is selected in such a way to attain optimum bitumen content and to produce a density in laboratory equivalent to that ultimately obtained in pavement under traffic (Roberts et al 1996). Typically a compaction effort of 35, 50 or 75 blows is applied in Marshall Mixture design depending on the anticipated traffic loading.

Several studies have sought to determine the extent, to which various types of laboratory compaction simulate field conditions. Von Quintus et al, (1988) used creep tests conducted in diametral mode to compare field cores with laboratory specimens compacted using the Texas gyratory – shear compactor, the California kneading compactor, the mobile steel wheel simulator, the Arizona vibratory /kneading compactor, and the Marshall hammer. The investigators ranked compaction devices based on their abilities to consistently simulate the Engineering properties of field cores, as follows;

1) Texas gyratory – shear compactor
2) California kneading compactor and mobile steel wheel simulator,
3) Arizona vibratory/kneading compactor , and
4) Marshall's hammer.

Vallerga and Zube (1953) evaluated the effect of laboratory compaction method using Hveem stabilometer to measure mixture stability. They concluded that kneading compaction was more effective in obtaining high densities and stabilities than the other two methods. Furthermore, the asphalt content at maximum stability was less with kneading compaction than with other compaction methods in use at the time. Field densities from in – service pavements, under traffic for several years, were invariably larger than laboratory densities of freshly prepared mixtures. Accordingly, the investigators emphasized that the laboratory compaction method and compaction effort must produce specimens representative of pavements after conditioning by traffic loading.

Norhidayah et al (2012), used gyratory, vibratory and slab roller compaction methods on aggregate structure. Their study concluded that the aggregates near the edge of the specimen tend to form a circumferential alignment while the aggregates near the centre of specimen are randomly oriented. The study showed that coarse particles are mostly concentrated at the bottom compared to the top of a specimen.

In soils, compaction density is influenced by compaction effort and water content. Similar to soils, the compaction density of asphalt mixture is affected by the compaction effort and bitumen content. The density of an asphalt mixture increases with increase in bitumen content reaching the maximum density at optimum content after which it decreases as illustrated in Figure 2.7.

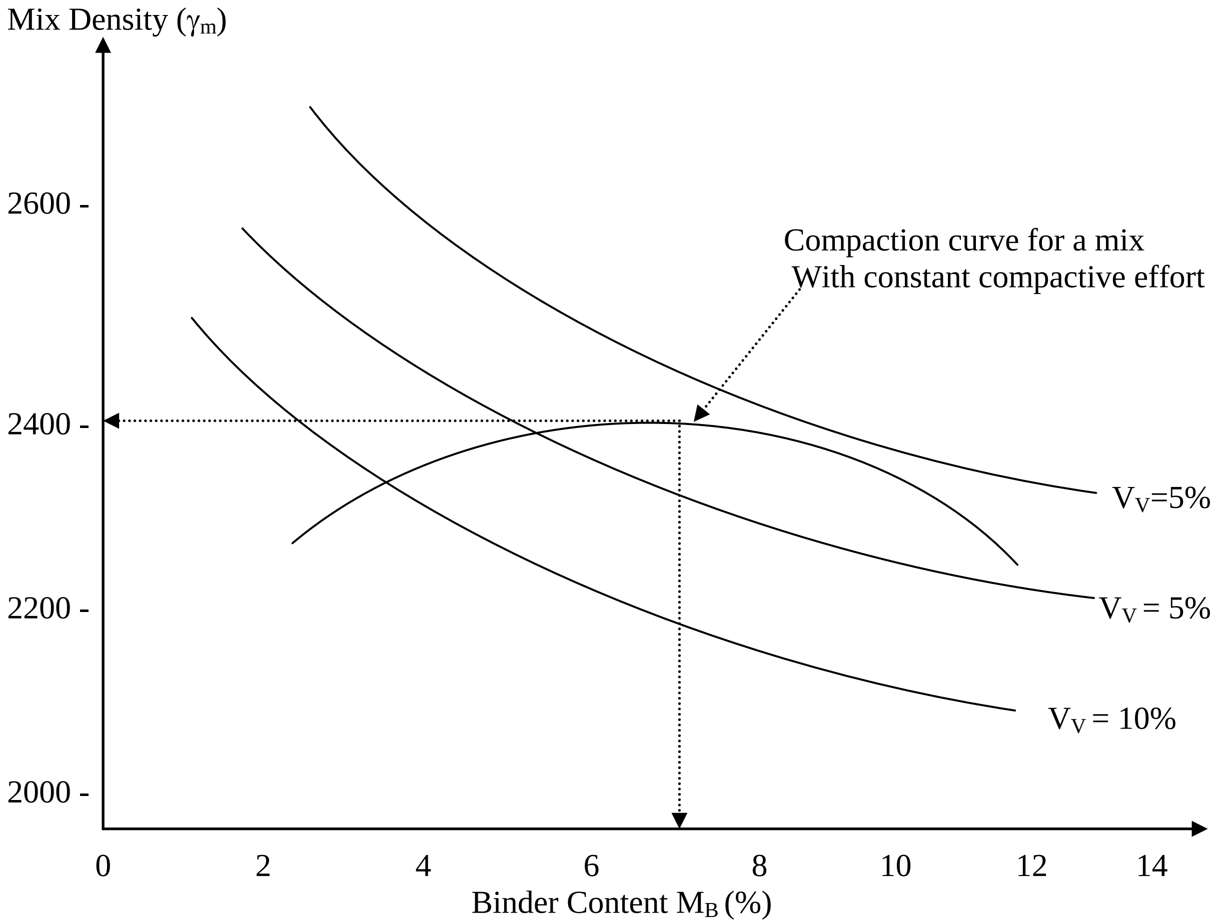


Figure 2.7; Effect of Bitumen Content on the Density of the Asphalt Mixture (Brown 1992)

2.4 Modeling of Permanent Deformation

Linear elastic multi-layer analysis in combination with laboratory based permanent deformation relationships and visco-elastic methods are some of the methods that are currently used to describe development of permanent deformation in asphalt mixtures. The term “hybrid models" is used to describe models that are a combination of elastic multi-layer analysis and laboratory based permanent deformation relationships. On the other hand, visco-elastic methods use time dependent material properties that are defined in terms of Maxwell and Kelvin elements, (Muraya, 2007).

Although many pavement models assume the properties within the pavement layers to remain constant irrespective of the stress distribution, stress dependency has been reported in literature. Several researchers have shown that the behaviour of unbound pavement materials and bound pavement materials such as asphalt mixtures is stress dependent (Antes, 2002). The consequence of these stress dependent behaviour is that material properties such as stiffness and poisson's ratio are not constant but vary as a function of the stresses at a particular point in a layer.

Visco - elastic models may provide a better description of the permanent deformation in asphalt mixtures. A viscoelastic material possesses both the elastic property of a solid and the viscous behaviour of a liquid, (Bahuguna et al, 1996). Suppose that a material is formed into a ball. If the ball is thrown on the floor and rebounds, it is said to be elastic. If the ball is left on the table and begins to flow and flatten gradually under its own weight it is said to be viscous. The viscous component makes the behaviour of viscoelastic materials time dependent: the longer the time, the more the material flows. Hot mix asphalt (HMA) is a viscoelastic material whose behaviour depends on the time of loading, so it is natural to apply theory of viscoelasticity to the analysis of layered systems. The latter theory is based on the elastic – viscoelastic correspondence principle by applying the Laplace transform to remove the time variable (t) with a transformed variable (p), thus changing a viscoelastic problem to an associated elastic problem. The Laplace inversion of the associated elastic problem from the transformed variable (p) to the time variable (t) results in the viscoelastic solutions.

Two general methods are used to characterize viscoelastic materials, (Bahuguna et al, 1996), one by mechanical model, and the other by creep compliance curve. The latter is used in VESSYS and KENLAYER models because of its simplicity, (Huang, 2004). The poisson's ratio, μ, has a relatively small effect on pavement behaviour and it is therefore assumed to be elastic independent of time. Therefore, only the modulus E is considered to be viscoelastic and time dependent.

2.4.1 Viscoelastic models

Viscoelastic models consist of mechanical, creep compliance and VESYS models. Mechanical models are formed of two basic elements: a spring and a dashpot. Various models which comprise mechanical models include; basic models, Maxwell model, Kelvin model, Burgers model and generalized model, (Kaliti, 2007).

Basic models characterize an elastic material using a spring with the material obeying Hooke's Law which asserts that stress is proportional to strain. A Maxwell model will characterize a material as a spring and a dashpot in series as shown in Figure 2.8(a). The theory assumes that under constant stress, the total strain is the sum of the strains of both spring and dashpot. On the other hand, the Kelvin model is a combination of spring and dashpot in parallel as shown in Figure 2.8(b). Both the spring and a dashpot have the same strain, but the total stress is the sum of the two stresses. Burgers model is a combination of Maxwell and Kelvin models in series as shown in Figure 2.8(c). The total strain is composed of three parts: an instantaneous elastic strain, a viscous strain, and a retarded elastic strain. The generalized model explains the effects of load duration on pavement responses .Under a single load application, the instantaneous and retarded elastic strains predominate, and the viscous strain is negligible. However , under a large number of load repetitions, the accumulation of viscous strains is the cause of permanent deformation.

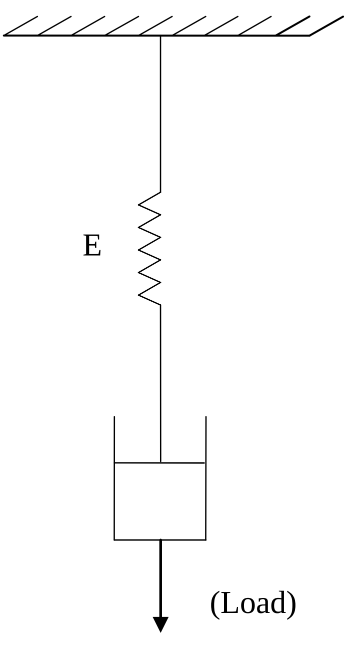


Figure 2.8(a); Maxwell Simple Viscous Model

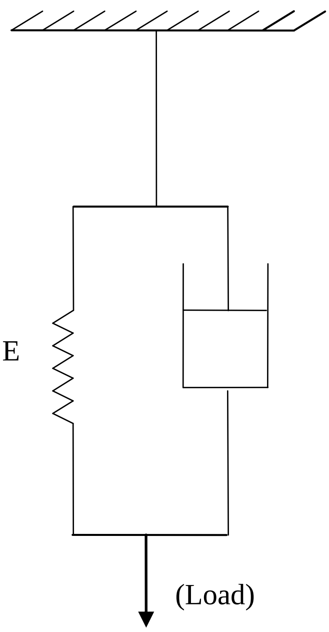


Figure 2.8(b); Kelvin Simple Viscous Model

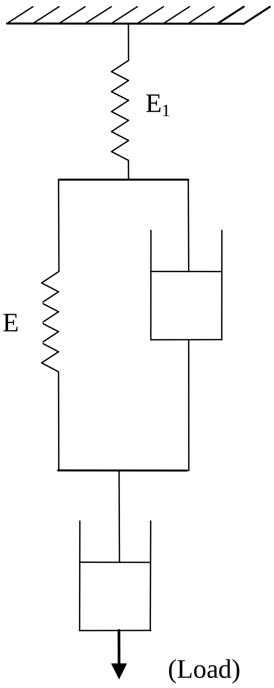


Figure 2.8(c); Burger's (Four –Element) Model

Creep compliance models characterizes viscoelastic materials at various times, D (t), defined as shown in Equation 2.1

$$D(t) = \varepsilon(t)/\sigma \quad \text{2.1}$$

In which $\varepsilon(t)$ is the time – dependent strain under a constant stress.

Under a constant stress, the creep compliance is a reciprocal of the Young's Modulus. For the generalized model, the creep compliance can be expressed as shown in Equation 2.2

$$D(t) = 1/E_0\,[1+t/T_0] + \sum 1/E_i\,[1-\exp(-t/T_i)] \quad \text{2.2}$$

In which T_0 is the relaxation time.

Given the various viscoelastic constants, E_0, T_0, E_i, and T_i for a generalized model, the creep compliances at various times can be computed from Equation 2.2.

The main difference between the elastic and visco-elastic approaches is that in the elastic theory the material is visualized to have fixed time-independent stress to strain ratios (Ashton and Moavenzadeh, 1967). The time-dependent material properties in the visco-elastic approach are defined in terms of Maxwell or Kelvin elements. It is suggested in literature that, under small strains, asphalt mixtures exhibit linear visco-elastic behaviour and that the visco-elastic theory may give a better prediction than the elastic theory for displacements in asphalt pavements (Huang, 1967). An important advantage of this approach is that moving wheel loads can be considered directly. This results in the correct time-rate of loading to be applied to each material element. While non - linear visco-elastic response characteristics may provide a more realistic estimate of pavement response, the associated mathematical complexities have limited past analysis (Sousa et al ,1991).

One of the better known visco-elastic models is VESYS (Kenis, 1997). VESYS is a visco-elastic probabilistic computer program for predicting pavement performance in terms of rutting, roughness and crack damage. In the VESYS model, the pavement layers are described by elastic or visco-elastic properties and the material properties are assumed to be isotropic.

The permanent deformation is calculated based on the laboratory determined permanent deformation Equation 2.3 that relates the load applications and recoverable strain to permanent deformation.

$$\Delta e_p (N) = e\mu N^{-\alpha} \quad \text{……………………………………………………..…2.3}$$

Where;

Δe_p = Vertical permanent deformation at the N^{th} load repetition.

e = Peak strain for a haversine load pulse of duration 0.01 sec measured on the 200^{th} repetition

μ,α = Model parameters

VESYS method predicts the rut depth based on the assumption that permanent strain is proportional to the resilient strain as shown by Equation 2.4

$$\varepsilon_p (N) = \mu\varepsilon N^{-\alpha} \quad \text{…………………………………………….……………2.4}$$

Where;

$\varepsilon_p (N)$ is the permanent or plastic strain due to single load application i.e. at the N^{th} load application

ε is the elastic or resilient strain at the 200th repetition

N is the load application number

μ is a permanent deformation parameter representing the constant of proportionality between permanent and elastic strains

α is a permanent deformation parameter indicating the rate of decrease in permanent deformation as the number of load applications increases.

The total permanent deformation can be obtained by integrating Equation 2.4 above as follows

$$\varepsilon_p = \int_0^N \varepsilon_p(N)\, dN = \varepsilon \mu \frac{N^{1-\alpha}}{1-\alpha} \quad \ldots\ldots 2.5$$

Equation 2.5 indicates that a plot of log ε_p versus log N results in a straight line as shown in Figure 2.9. From Equation 2.5,

$$\text{Log}\varepsilon_p = \log(\varepsilon \mu / (1-\alpha)) + (1-\alpha) \log N \quad \ldots\ldots 2.6$$

So the slope of the straight line is $S = 1-\alpha$, or

$$\alpha = 1 - S \quad \ldots\ldots 2.7$$

The intercept occurs at N=1, $1 = \varepsilon \mu / (1-\alpha)$, so

$$\mu = S / \varepsilon \quad \ldots\ldots 2.8$$

To determine the permanent deformation parameters of the layer system, α_{sys} and μ_{sys}, from those of individual layers, it is further assumed that the sum of permanent and recoverable strains due to each load application is a constant and equals the elastic strain at the 200th repetition. This means that after the 200th repetition,

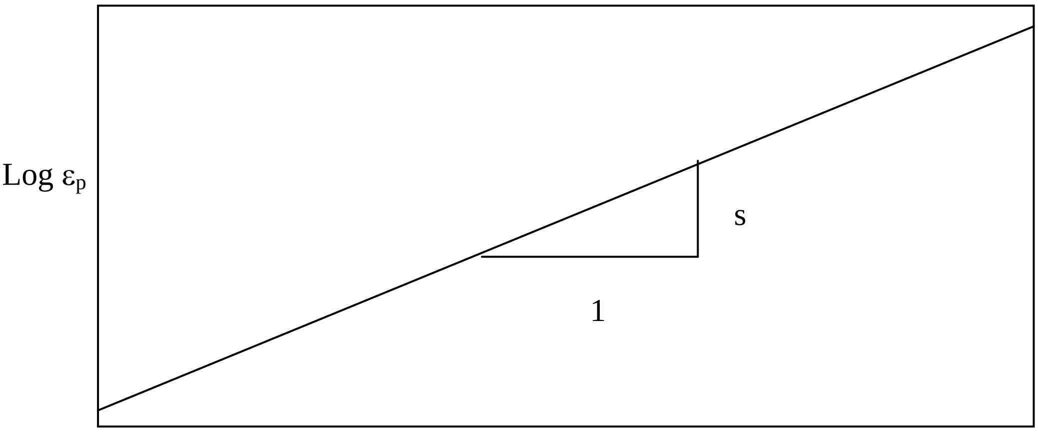


Figure 2.9; log load duration vs. log ε_p

$$\varepsilon = \varepsilon_p(N) + \varepsilon_r(N) \quad \ldots\ldots 2.9$$

In which ε_r (N) is the recoverable strain due to each load application. Substituting Equation 2.4 into Equation 2.9 yields

$$\varepsilon_r(N) = \varepsilon(1 - \mu N^{-\alpha}) \quad \ldots\ldots 2.10$$

Under the same stresses, strains are inversely proportional to the moduli so Equation 2.10 can be written as

$$\varepsilon_r(N) = E/(1 - \mu N^{-\alpha}) = \frac{EN^{\alpha}}{N^{\alpha} - \mu} \quad \ldots\ldots 2.11$$

In which ε_r (N) is the elastic modulus due to unloading and E is the elastic modulus due to loading. Note that ε_r (N), which is the unloading modulus for each individual layer, is not a constant but increases with the increase of load repetitions. These unloading moduli are used to determine the recoverable deformation w_r (N) at different values of N. The permanent deformation w_p (N) can then be computed by

$$w_p(N) = w - w_r(N) \quad \ldots\ldots 2.12$$

In which w is the elastic deformation due to loading similar to Equation 1, w_p (N) can be expressed by

$$w_p\,(N) = \mu_{sys} w N^{-\alpha sys} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; 2.13$$

Combining Equation 2.12 and 2.13 gives

$$1 - w_r\,(N)/w = \mu_{sys}\, N^{-\alpha sys} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; 2.14$$

Equation 2.14 shows that a plot of log[1 - w_r (N)/w] versus log N results in a straight line as shown in Figure 2.10 where the slope of a straight line is α_{sys} and the intercept at N = 1 is log μ_{sys}

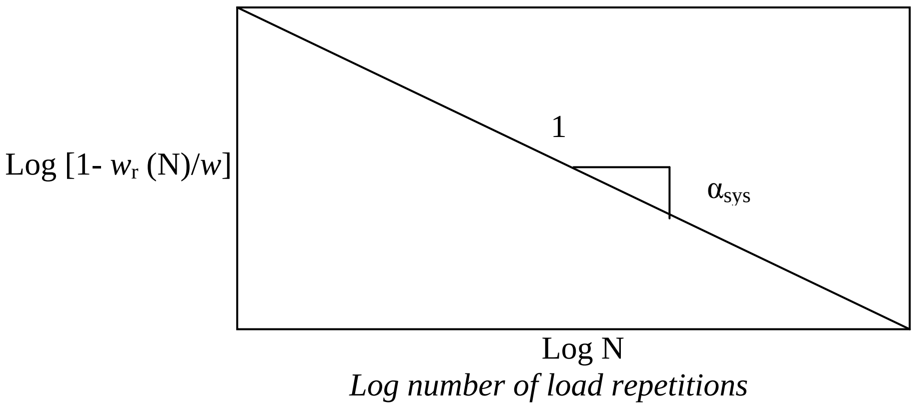


Figure 2.10; Log-log plot of [1- w_r (N)/w] versus number of load Repetitions (Huang, 2004)

The determination of α_{sys} and μ_{sys} from the α and μ of each layer can be summarized as follows:

i. Assume several values of N, say 1, 10^2, 10^3, 10^4, 10^5, and determine the unloading modulus ε_r (N) of each layer by Equation 2.11.
ii. Using the unloading moduli as the elastic moduli, determine the recoverable deformation w_r (N) at the surface by the layered theory.
iii. For each N, compute 1 - w_r (N)/w and plot it against N on log scales.
iv. Fit the plotted points by a least squares line. The slope of the line is α_{sys} and the intercept at N = 1 is $\mu_{sys.}$
v. Determine the permanent deformation by Equation 2.12.

2.4.2 Plastic Models

In order to arrive to a better understanding of the damage mechanisms in asphalt concrete, it is necessary to establish more fundamental knowledge about the materials properties and response. In studying the response, influence factors such as temperature and strain rate must be considered. Furthermore, it must be realized that to truly investigate material behaviour, structural influences on observed response should be prevented. For example, tests should be designed in such a way that uniform internal states of stress are obtained (Molenaar et al 1998). Examples of plastic models include the Desai and Unified models.

Briefly the Desai flow surface can be schematized as shown in Figure 2.11. These figures compare the one- and two- dimensional situations; originally, the behaviour is linearly elastic represented by straight line until point 1 in the left hand diagram. From point 1, which corresponds to ellipse 1 in the 2D case, the response is non linear but the load carrying capacity can still increase. This response is commonly known as hardening and in the 1D case it looks like a curve with diminishing slope, (between parts 1 and 3 in the 2D case, this phase of response corresponds to a series of successive, incremental ellipses 1 to 3). The strength (the maximum, point 3) in the 1D situation corresponds to the largest ellipse in the 2D case. After this point the strength decreases and softening is initiated. This is illustrated by descending branch in the 1D diagram (between point 3 and 4). In the 2D case, this gain corresponds to a series of successive ellipses, this time diminishing in size (Molenaar et al 1998).

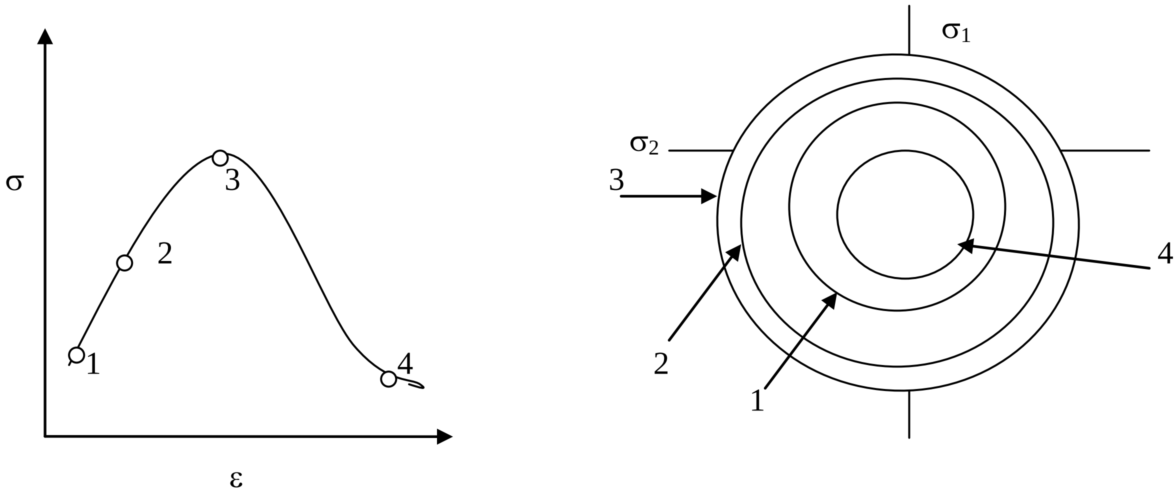


Figure 2.11 Schematization of the Desai flow surface for the 1- and 2- dimensional case. The numbered points in the 1D diagram correspond to the ellipses in the 2D case. (Molenaar et al 1998).Sources; Ref. (2)

In the three dimensional case; the representation of the stress conditions for a certain state to material response (e.g. failure, initiation of plasticity etc) is a bit more complex. As evident in the ACRe model (Erken's 2002), the Desai flow surface is also applicable to the three dimensional case. Figure 2.11 shows the Desai flow surface. The expression for this surface is given by Equation 2.15.

$$f=J_2/P^2_a- [-\alpha((I_1+R)/P_a)^n + \gamma((I_1+R)/P_a)^2]/ (1-\beta\cos3\theta)^{1/2} = 0 \quad \text{2.15}$$

Where; f = Desai flow surface

$$I_1=\sigma_{xx}+\sigma_{YY}+\sigma_{ZZ} \quad \text{2.16}$$

$$J_2=1/2(S^2_1+ S^2_2+ S^2_3)=1/6[(\sigma_{xx}-\sigma_{YY})^2 + (\sigma_{YY}-\sigma_{ZZ})^2+ (\sigma_{ZZ}-\sigma_{XX})^2] + \tau^2_{XY}+\tau^2_{YZ}+\tau^2_{XZ} \quad \text{2.17}$$

$$J_3= (\sigma_{xx}- P) (\sigma_{YY}- P) (\sigma_{ZZ}- P) +2\tau_{XY}\tau_{YZ}\tau_{XZ} - (\sigma_{xx}-P) \tau^2_{YZ}- (\sigma_{YY}- P) \tau^2_{XZ}-(\sigma_{ZZ}- P) \tau^2_{XY} \quad \text{2.18}$$

$$P= (\sigma_{xx}+\sigma_{YY}+\sigma_{ZZ})/3 \quad \text{2.19}$$

$$\text{Cos}3\theta = 3/2*3^{1/3}* J_3/ (J_2)^{3/2} \quad \text{2.20}$$

Where;

I_1= the first stress invariant

J_2 = the second deviatoric stress invariant

J_3 = the third deviatoric stress invariant

P = Isotropic stress

S_i = i^{th} principal deviator stress

P_a = 0.1[Mpa]

α, γ, n and R =Model parameters

The model parameters α, γ, n and R govern specific aspects of the surface. These parameters are dependent on the test conditions such as temperature, strain rate, confinement or any other test variable. A description of the role played by each of the parameters with respect to the stress conditions represented by means of the I_1 -SQR J_2 stress invariants.

The Unified model applies a time temperature superposition principle, a common process in developing master curves for the complex modulus of the asphalt mixtures. Until recently, this principle has not been used to quantify strength and failure characteristics of the asphalt mixtures. In his study, Muraya (2007) used the unified model (Medani 2006) manually as a tool to quantify the properties of asphalt mixtures in relation to time temperature dependency. The main advantage of this model lies in the fact that it can be used to describe a wide range of properties such as stiffness, compressive strength and tensile strength in relation to temperature and time. The model also incorporates theoretical maximum and minimum limits of the property under consideration. The implications of this model are as follows:

a. The temperature susceptibility can be obtained from tests that are relatively easy to conduct.
b. Feasible estimation of minimum and maximum limits of desired material properties.
c. Construction of master curves for different material properties, for example stiffness, compressive strength and tensile strength.

The unified model is expressed as follows;

$P = P_{high} + (P_{low} - P_{high})\ S$ ……………………………………………………………..2.21

Where;

$S = e^{(-[\mu,\beta]\ 2}$ ……………………………………………………………………… 2.22

Where;

$\beta = e^{(-r,\ [T-T_0]}$

P=Property under consideration

P_{high} = Value of the property as the time derivative variable approaches infinity.

P_{low} = Value of property as the time derivative variable approaches zero.

T_o = A temperature reference

T = Temperature.

μ, β, S = constants

The unified model is used to capture the effect of strain rate and temperature in the properties of asphalt mixtures. Nevertheless, the parameters of this model can be modified to describe the effect of confinement on the aggregate skeleton. Application of the same model in the description of the mixture component, allows a similar approach to be used in the characterization of the different components.

2.4.3 Linear Elastic Models

A good example of the linear elastic model is the hybrid models. Hybrid models describe the development of permanent deformation in asphalt mixtures as illustrated in Figure 2.12. The stresses or strains calculated from linear elastic multi-layer analysis are used in laboratory based on permanent deformation relationships to predict permanent deformation. The laboratory based permanent deformation relationships are developed by means of laboratory tests such as creep, repeated load, uniaxial or triaxial tests (Adrian et al, 2011).

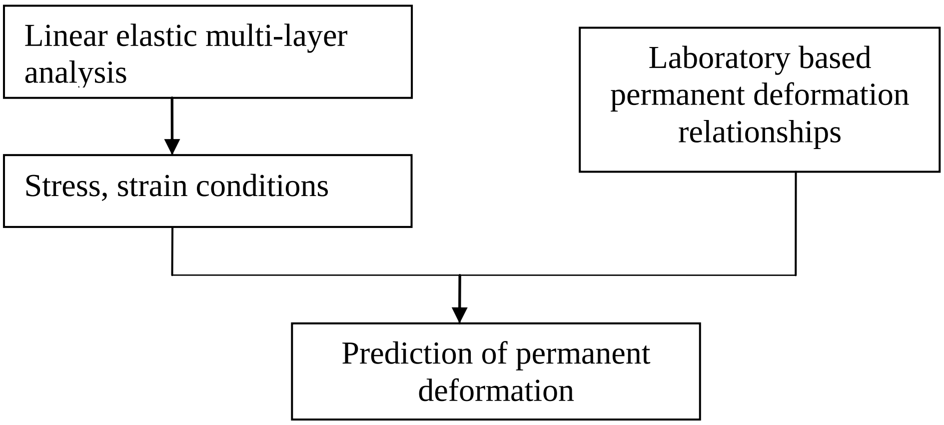


Figure 2.12; Hybrid models (Muraya, 2007)

In 1885, Boussinesq developed a theory for the calculation of stresses and displacements based on a concentrated load applied on an elastic half-space (Huang, 1993). In his theory, the stresses were dependent on the load, depth and distance while the displacements were a function of load, depth, distance, poisson's ratio and elastic modulus. Alvin and Ulery extended Boussinesq theory to allow for the determination of stress, strain and deflection at any point in a homogenous half space subjected to a vertical circular load of uniform pressure. Odemark developed the equivalent layer thickness theory for application of the Boussinesq theory to multilayer systems with varying elastic moduli. However, the combined Odemark and Boussinesq theory could not predict horizontal tensile stresses that can be generated if a stiff bound layer is applied on a relatively softer layer (Molenaar, 1998). Since the occurrence of tensile stresses has been identified among the causes of failure in asphalt layers, founded on unbound materials, researchers such as Nijboer (1955) and Huang (1993) have come up with procedures to determine the tensile stresses occurring at the bottom of the bound layers based on a circular uniformly distributed vertical contact stress. Burmister developed the linear elastic multi-layered theory, which is considered to give reasonable estimates of the stresses and strains occurring in bound materials (Molenaar, 1998).

Hybrid models characterize the pavement layers with elastic materials properties. In these models, each layer is characterized by single elastic and poisson's ratio parameter and the stresses in the pavement layers are calculated by means of elastic analysis (AASHTO, 2013). The single elastic parameters do not account for the stress dependency nor do they account for time – dependent visco-elastic material properties in the asphalt layers. The wheel load is represented as a static load and the contact stress distribution is generally considered to be uniformly distributed over a regular (mostly circular) contact area.

One example of hybrid model is the Esso (Eckmann 1987) method. In this method, the permanent deformation is determined as a function of the pavement stresses and permanent deformation relationships as developed from repeated load triaxial tests. The permanent deformation in the pavement is calculated based on Equations (2.23) and (2.24) below;

$$\varepsilon_i = \alpha t_i^{\beta} \quad \ldots\ldots\ldots \ 2.23$$

$$\varepsilon = \varepsilon_i + \varepsilon_j \Delta t \quad \ldots\ldots\ldots \ 2.24$$

Where;

ε_i=Initial permanent deformation

α, β=Regression parameters dependent on type of mixture and test conditions.

t_i =Initial loading time

ε = Permanent deformation.

ε_j= Rate of permanent deformation dependent on type of mixture and test conditions.

Δt = Change in loading time.

The hybrid model approach is attractive because of its simplicity and straight forwardness. However, literature review suggests that the agreement between the predicted and the actual permanent deformation varies from very good to very poor (Sousa et al, 1991).

2.4.4 Laboratory Based Permanent Deformation Relationships

Laboratory based permanent deformation relationships mainly include constitutive laws (Swedish National Road Administration, 2010). Constitutive laws may take several forms like logarithmic relationships, power relationships, exponential relationships among others.

The natural logarithmic law is of the form shown by Equation 2.25 while the power law takes the form shown by Equation 2.26.

$$\ln C_1 = a + b\ln T \qquad 2.25$$

$$C_1 = aC_2^b \qquad 2.26$$

Where;

C_1, C_2 = strength characteristic

T = temperature (^0C)

a,b = Material constants

2.5 Influence of Asphalt mixture Components on Permanent Deformation Behaviour

Higher permanent deformation occurs in asphalt mixtures at high temperatures compared to low temperatures. Brown and Snaith (1974) suggested that the increase in deformation is related to the decrease in binder viscosity at high temperatures (40^oC), thereby leading to a lower interlock between the aggregates. The contribution of aggregates skeleton towards the behaviour of mixture becomes more significant at higher temperatures. Pellinen and Witczack (2002) found the aggregate influence to be more dominant than that of the binder on the modulus at high temperatures and the binder influence to be more dominant over the aggregate influence at low temperatures. At high temperatures, the effects of confining stresses play a significant role in permanent deformation of the mixture. Awad (1972) found that the effects of confining stresses on elastic and permanent deformation strains respectively was more important at high temperatures than at low temperatures.

From the above discussion, it is demonstrated that both the aggregates and the mortar play a crucial role towards the resistance to permanent deformation in asphalt mixtures. This is so especially at high temperatures where the role of aggregate skeleton becomes dominant;

asphalt mixtures have many similarities with unbound materials. It is therefore reasonable to assume that failure mechanisms that are applicable to unbound materials are valid for asphalt mixtures as well. The Mohr Coulomb criterion is widely used to describe the shear characteristics of unbound materials and asphalt materials (Molenaar 1993). Plasticity models such as the Mohr-Coulomb yield criterion to describe the response of a material in relation to an ultimate surface beyond which no stresses are permitted to occur. If the state of stress is inside the ultimate surface, the deformations are purely elastic and plastic deformation occurs at states of stress on ultimate surface.

The Mohr-Coulomb failure criterion is expressed by Equation 2.27 below;

$$\tau = C + \sigma \tan \phi \qquad \text{..........} \; 2.27$$

Where;

τ = Shear strength

C= Cohesion

σ = Normal stress

ϕ = Angle of internal friction

The equation is an elegant means of demonstrating the effect of both aggregates and mortar towards the resistance of permanent deformation behaviour. Figure 2.13 illustrates the effect of aggregates and the mortar on failure behaviour of asphalt mixtures. For simplicity purposes, the cohesive strength is considered to be wholly influenced by the mortar while the angle of internal friction is influenced entirely by the aggregates. If the binder has a high cohesion at high temperatures, then it will provide better resistance to permanent deformation than a binder with a lower cohesion. Similarly aggregates with a higher internal angle of friction can be expected to provide a better resistance to permanent deformation than similar aggregates with a lower angle of internal friction.

However, it must be emphasized that this is a very simple idealization of an asphalt mixture. The cohesion strength of the mortar is dependent on strain rate as well as temperature. It may well be that one mortar may seem to be better at a given strain rate and temperature but the opposite can be true at a different set of strain rate and temperature in comparison to another mortar. Another reason is that permanent deformation may involve rotation of aggregate particles; a fact that cannot be represented by the internal angle of friction.

Nevertheless, simple examples illustrate that both the mortar and aggregate skeleton play a role on the resistance to permanent deformation. The extent of a role they play depends on the type of mixture and loading conditions. For a good understanding of permanent deformation behaviour it is important to understand the role of the material components. However, little or no quantitative information is available on how the various components of an asphalt mixture contribute towards the resistance to permanent deformation.

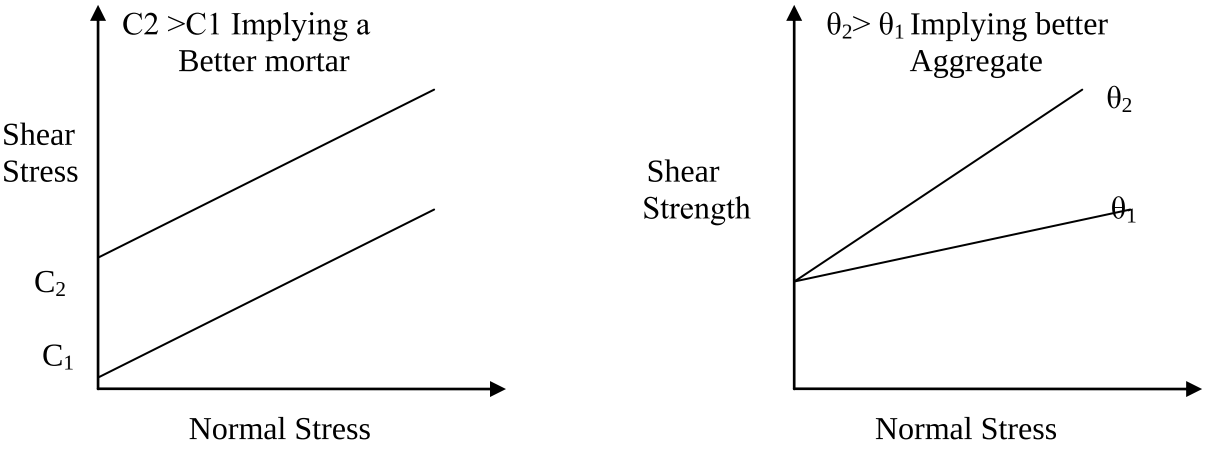


Figure 2.13; Effects of Aggregates and Mortar on the Failure Behaviour of Asphalt mixture (Source; Muraya, 2007)

2.6 Model to be used in this Research

VESYS model as discussed in section 2.4.1 assumes that permanent strain is proportional to resilient strain and that the sum of the permanent and recoverable strains due to load application is constant. The sum of permanent and recoverable strains is not constant due to other factors which affect permanent deformation of asphalt mixtures such as the loading temperature, aggregate gradation and bitumen content. This means that the model is not able to simulate the real pavement conditions and hence not suitable to characterize permanent deformation of asphalt mixtures.

Hybrid models characterize pavement layers with elastic material properties and hence they fail to account for both stress and time dependency of a viscoelastic material properties.

Literature review suggests that agreement between the permanent deformation predicted using hybrid models and actual permanent deformation is poor, (Sousa et al 1991) and hence they may not be appropriate to model permanent deformation of asphalt mixtures.

In his study, Muraya (2007) found that viscoelastic models are not sufficient for the purposes of describing permanent deformation of asphalt mixtures. Muraya (2007) further concluded that plasticity models as described by Medani (2006) may provide better description of the permanent deformation in asphalt mixtures. The materials that exhibit elastic characteristics rebound back to their original shape after the stress removal. In other words, they return all the energy that had been stored in the form of strain energy. On the other hand, materials that display plastic behaviour recover some of the final deformation and retain the balance as permanent deformation. The study assumed that the asphalt material to be tested behaves elastically up to a certain point after which they start behaving plastically until complete failure is achieved. This is a case where say the road base is made of dense bitumen macadam and asphalt concrete layer on the surface i.e. base course and wearing course, respectively. Since, dense bitumen macadam is stiffer than the asphalt concrete, the asphalt concrete is likely to spread laterally on top of the DBM layer due to the traffic loading effects.

From the discussion in the preceding paragraphs, the permanent deformation behaviour of asphalt mixtures is best described by the Desai flow surface as earlier captured in section 2.4.2. Graphic illustration is as already shown in Figure 2.11. The two components compare the one- and two- dimensional situations; originally, the behaviour is linearly elastic represented by straight line from the origin until point 1 in the left hand diagram. The gradient of the linear part of the curve is described as the *tangent stiffness*. From point 1, which corresponds to ellipse 1 in the 2D case, the response is non linear but the load carrying capacity can still increase. This response is commonly known as hardening and in the 1D case it looks like a curve with diminishing slope, (between parts 1 and 3 in the 2D case, this phase of response corresponds to a series of successive, incremental ellipses 1 to 3). The strength (the maximum, point 3) in the 1D situation corresponds to the largest ellipse in the 2D case. The vertical stress at point 1 is described as the *stress at initiation of plasticity*. After this point the strength decreases and softening is initiated. This is illustrated by a descending branch in the 1D diagram (between point 3 and 4). In the 2D case, this gain corresponds to a series of successive ellipses, this time diminishing in size (Molenaar et al 2002). The vertical stress at point 2 is described as the *stress at initiation of dilation.* The vertical stress at point 3 is referred to as the *maximum stress* which is equivalent to the specimen's *shear stress* at failure or *compressive strength*.

Constitutive laws as described by equations 2.25 and 2.26 are a form of laboratory based relationships for permanent deformation of asphalt mixtures and will be used in the study to model permanent deformation of asphalt mixtures. The constitutive equations will be used to relate the various parameters which will be tested in the study. Constitutive equations or laws in the context of this study refer to correlations between pairs of measured quantities or characteristics of asphalt mixtures. The quantities or mixture characteristics will be determined using the triaxial compression test. The various characteristics of the asphalt mixtures in this study are to be determined at different temperature levels i.e. 20, 30, 40, 50 and 60^0C.

Constitutive laws may take several forms like natural logarithmic relationship, logarithmic relationships, power relationships, exponential relationships among others. In this study two forms of constitutive laws are used namely the natural logarithmic law and the power model law. Natural logarithmic law is used to relate the tangent stiffness with the loading temperature while the power law is used to model the maximum vertical stress in respect to the confining stress. The natural logarithmic law is of the form shown by Equation 2.28 while the power model takes the form shown by Equation 2.29

$$\mathrm{Ln}T_S = C_1 + k_1 \ln T \qquad \text{2.28}$$

$$f_{ca} = a\sigma_3^b \qquad \text{2.29}$$

Where;

T_S = Tangent stiffness

T = Loading temperature (^{0}C)

f_{ca} = Maximum stress

σ_3 = Confining stress

C_1, k_1, a, b = Material constants

2.7 Literature Review Summary

Kenyan Road Designer Manual part III of 1987 is used to design the thickness of asphalt pavement layers as well as the materials composition. The design method in the manual assume pavement to be an elastic multi –layer system in which materials are characterized by Young's modulus of elasticity and poisson's ratio. Pavement materials are assumed to be homogenous and isotropic and traffic is expressed in cumulative numbers of repetitions of a standard load. The maximum design axle load is taken as 130kN. The loading time is taken 0.02 seconds equivalent to 60km/h with the pavement loaded at a temperature of 18^0C which is the weighted annual mean for Nairobi.

On the outer lane of pavement's upgrade section, heavily loaded vehicles are found to move at slow speeds way below the one assumed by the Road designers manual (60km/h). Vehicles especially the commercial trucks are found to be overloaded and hence exceeding a maximum of 130kN axle load allowed for in the Road designers manual. The loading temperatures in Kenyan roads are found to range between 20^0C and 60^0C way beyond the 18^0C allowed for in the Road designers manual.

The foregoing paragraph identifies the gaps that exist in our current Road Designers Manual part III of 1987. It is in the interest of this study to try and bridge the above identified gaps. To bridge the gap due to the design speeds of slow moving trucks on upgrade sections of pavement, the study tested asphalt samples at static speeds which is almost similar to speeds of slow moving trucks. To bridge the gap that exist due to loading temperatures asphalt samples are tested at temperatures ranging from 20-60 ^{0}C equivalent to actual temperatures on Kenyan roads. To overcome the gap which exists between the maximum axle loads (130kN) and the actual loading on pavements, varying loadings were applied to the sample including the confining stresses.

Chapter three

Characterization of Pavement Materials for Permanent Deformation

3.1 Introduction

Rutting in paving materials develops gradually with increasing number of load applications, usually appearing as longitudinal depressions in the wheel paths accompanied by small upheavals to the sides. It is caused by a combination of densification (decrease in volume and, hence, increase in density) and shear deformation and can occur in any one or more of the pavement layers as well as in the subgrade. Trenching studies performed at the AASHO Road Test (Highway Research Board, 1962) and test – track studies reported by Hofstra and Klomp (1972) indicated that shear deformation rather than densification was the primary rutting mechanism. The importance of placing materials at high densities in order to minimize shear deformation was therefore emphasized.

Hofstra and Klomp (1972) found that the deformation through the asphalt – concrete layer was greatest near the loaded surface and gradually decreased at lower levels. Because rutting is caused by plastic flow, such a distribution of rutting with depth is reasonable: more resistance to plastic flow is encountered at greater depths and shear stresses are smaller there as well.

Uge et al (1974) reported that the deformation within an asphalt layer (thickness reduction under the action of pneumatic tyres no longer increased with increasing layer thickness beyond a certain threshold (13cm in their case). Measurement at the AASHO Road Test (Highway Research Board, 1962) indicated that the surface rut depth reached a limiting value for asphalt concrete thickness of approximately 10 inches. Thicker layers did not exhibit additional rutting. These results strongly suggest that, at least for reasonably stiff supporting materials, most pavement rutting is confined to the asphalt concrete layer.

Various parameters are measured in order to assist in defining the behaviour of asphalt mixtures being tested for permanent deformation. These parameters are as defined in section 3.2.

Plate 3.1 is an example of permanent deformation of asphalt pavements in form of rutting on an upgrade section frequented by heavy trucks moving at a slow speed.

(a).The photo on the left shows permanent deformation in form of rutting on the outer most lane of a steep road. The photo on the right shows a six axle lorry moving at a snail speed on an outer lane of a steep road.

(b).The photo on the left shows the effect of a heavy load applied at a slow speed hence increasing duration of loading and therefore an increased effect on the pavement as evident on the right photo. Notice an intact inner lane where light vehicles pass at a higher speed hence reduced loading duration.

(c).The photo on the left shows speeding light vehicles using inner lane where permanent Deformation in rut form is minimal and heavy vehicles using the outermost lane at a slow Speed with permanent deformation in form of a rut dominating the lane. The right photo Shows a rut which has continued to be loaded by heavy vehicles over time and as a result Cracks have developed.

Plate 3.1; Photos illustrating permanent deformation of asphalt mixtures in Kenya in form of Rutting

3.2 Definitions

3.2.1 Modulus of Deformation/Stiffness Modulus

The term modulus of deformation of an engineering material is generally used to refer to the gradient of its stress-strain behaviour response. However, in modern engineering the reference taken by the term depends on the stress – strain characteristics of the materials in question, for example, whether material is in elastic or viscoelastic state.

In elastic material behaviour, the modulus of deformation is referred to as modulus of elasticity or elastic modulus of deformation or Young's modulus of elasticity. In this elastic state, the stress-strain relationship is linear and as such, Hooke's law is obeyed.

Viscoelastic behaviour response, as exhibited by some flexible pavement materials has a curvilinear relationship between stress and strain. Hence, stress and strain relationship can only be described at a given stress because of its varied nature. The gradient of this varied relationship at any given stress value on the strain-stress curve is most appropriately referred to as *modulus of deformation*. Pavement materials which exhibit viscoelastic stress-strain characteristics include subgrade soils (cohesive and granular), unbound base and subbase granular materials and particularly bituminous mixtures.

From the above discussion, it is clear that Young's modulus of Elasticity applies only to elastic response of materials or those materials which behave nearly like so; whereas modulus of deformation is a more general term describing the gradient of stress-strain curve. The elastic behaviour response of pavement materials is a special case of more general viscoelastic response, which depends on the rate of loading and the pavement temperature. This is particularly true in bituminous mixtures where the stress-strain behaviour is a function of loading time and temperature. The modulus of deformation that is a function of loading time and temperature is called *stiffness modulus*.

Modulus of deformation has got different sub-definitions depending on the loading rate, or loading time expected (Kaliti, 2007). When loading time is 1 second or more, this loading condition is termed static. When loading time is less than 1 second, the loading condition is termed dynamic. Static and dynamic tests respectively are then used in characterizing materials. As such, there are four sub-definitions of moduli of deformation that are applicable to static loading condition. These include tangent modulus of deformation, cord modulus of deformation, secant modulus of deformation and average modulus of deformation. In dynamic loading conditions, the modulus of deformation may be evaluated as the resilient modulus of deformation.

3.2.1.1 Tangent Modulus of Deformation

Tangent modulus is a slope of a tangent at some arbitrally chosen stress on a stress-strain characteristic curve. The magnitude of this quantity is constant only for a given stress-strain state, time of loading , temperature and material type, since pavement materials are subjected to varied stress-strain states, time of loading and pavement temperature. In situations where small stresses are expected, the initial tangent modulus is used although its estimation from stress-strain curve is difficult. Of interest to this study was the *tangent stiffness* modulus of deformation. This is the gradient of the linear part of the stress-strain curves for the various asphalt mixtures in this study tested at different loading conditions.

3.2.1.2 Secant Modulus of Deformation

Secant modulus is the slope of the cord drawn from origin of a stress-strain curve to some arbitrally chosen stress. The secant modulus would appear to be the most useful modulus of deformation in pavement material analysis. This is especially so when it is evaluated between the origin and failure point. In this case, it gives a theoretical value as the stress-strain curve that is between the value at origin and that value at the failure point. This intermediate value of secant modulus may be well placed to assess whether the material is under designed or over designed. Further, secant modulus of compressive and tensile stress may form a sound basis for structural pavement design. An illustration is given in figure 2.14(a-ii).

3.2.1.3 Average Modulus of Deformation

Average modulus of deformation is the slope of the least squares regression line of stress-strain characteristic. The average modulus is only reliable for engineering design when stress-strain values are within linear or elastic range. As behaviour becomes more and more viscoelastic the average modulus becomes more unrepresentative of the actual stress-strain behaviour of materials in question.

3.2.1.4 Cord Modulus of Deformation

Cord modulus of deformation is the slope of a cord drawn between two arbitrally chosen stresses on the stress-strain relationship. Cord modulus is the best suited for analysis of materials expected to be subjected to varied stress-strain states. However, its best application would be when small stress changes are expected. An illustration is given in figure 2.14(a-i).

3.2.1.5 Resilient Modulus of Deformation

Resilient modulus is the slope of relationship of applied stress and recoverable strain for any number of load applications. This is the most suited quantity to evaluate fatigue and resilient behaviour of pavement materials, especially in soils and granular materials, subjected to dynamic loading. An illustration is given in figure 2.14(a-iii).

3.2. 2 Tensile and Compressive Moduli of Deformation

Tensile modulus of deformation is the slope of the tensile stress-strain behavioral response of materials subjected to tensile stresses. The compressive modulus of deformation on the other hand is the gradient of compressive stress-strain relationship of materials subjected to compressive loading. It is found that the constituent materials of asphalt flexible pavements are subjected to tensile and compressive stresses by each wheel load that moves along the roadway. If a wheel load is imagined to make a depression on a flexible pavement, the material at the top most of the pavement which is into contact with the wheel is subjected to compressive forces, whereas the materials at the bottom layer of the pavement structure are subjected to tensile stresses.

In order for pavement material to be able to take up some tensile stresses, it must be bound up together by use of a binder. Therefore, only bituminous mixtures are able to take tensile stresses in flexible pavements. Cement and lime stabilized materials have low tensile strength. However, all pavement materials can take up some good magnitude of compressive stress. The deformations in pavement structures are of reversing sinusoidal by the multiple axle wheel loads of a vehicle. Therefore, the materials in a given point within the pavement structure are alternatively subjected to tensile and compressive stresses. Furthermore, the vertical stresses in a loaded pavement are compressive in nature.

The best pavement materials must be able to take tensile and compressive stresses. The repeatedly reversing sinusoidal stress form taken by the pavement structure on loading subjects the materials to fatigue. Such materials must therefore possess some good fatigue resistance properties. In addition to possessing tensile, compressive and fatigue resistances, bituminous mixtures have relatively high cohesion and internal friction under normal pavement surface temperatures. High shear strength is necessary if the materials are to have high internal stability. As such, there is high efficiency in lateral dissipation of applied wheel loads. Consequently, there is realized a protection of the subgrade and the materials within the pavement from excessive compressive stresses, which may exceed their compressive strengths.

3.2.3 Maximum stress

For the purposes of this study, maximum stress refers to the vertical stress at which an asphalt mix specimen fails under triaxial loading. Maximum stress is a measure of the extent to which an asphalt mixture can resist permanent deformation. The higher the magnitude of the maximum stress, the higher the resistance to permanent deformation and vice versa. It is to a large extent a measure of the compressive strength of asphalt mixtures.

3.2.4 Stress at Initiation of Plasticity

Stress at initiation of plasticity refers to the maximum vertical stress at the elastic phase in a stress-strain curve of an asphalt mix specimen under triaxial compression test. It is the vertical stress at which the asphalt mixtures start to undergo permanent deformation in form of plastic deformations. The higher the stress at the initiation of plasticity, the better the asphalt mix is in resisting permanent deformation. This parameter is expected to reduce with increase in the loading temperature as the asphalt mixtures losses strength due to increased inconsistency of the bitumen binders at high temperatures.

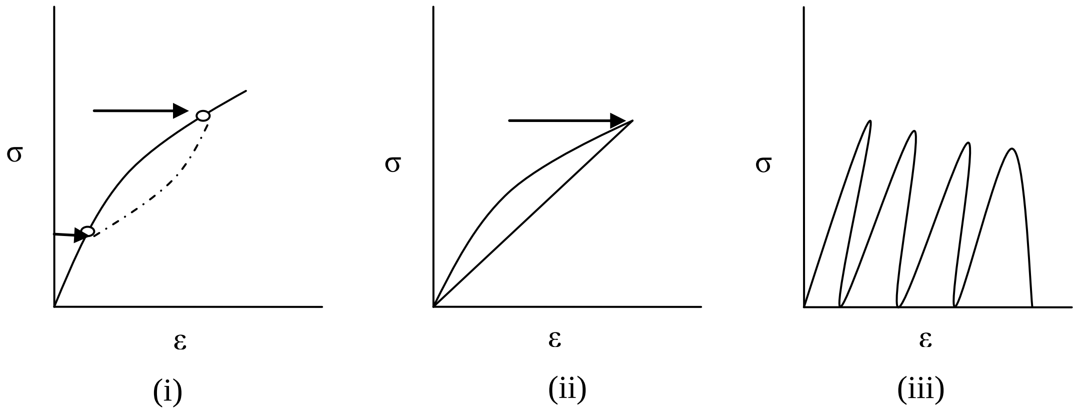


Figure 2.14: An illustration of cord modulus of deformation (i), secant modulus of deformation (ii) and resilient modulus of deformation (iii)

3.3 Tests for Permanent Deformation of Asphalt Mixtures

3.3.1 Introduction

Many essential fundamental tests are in use to evaluate static and dynamic tensile and compressive stress-strain relationships (i.e. moduli of deformation) of pavement materials. The tests are also able to measure the materials failure criteria in different modes of distress e.g. tensile strength, compressive strength, fatigue resistance, shear strength and permanent deformation resistance. The tests are meant to define input parameters for multilayered theory solutions. The various tests that are used in characterization of asphalt mixtures for permanent deformation are as described in sections 3.3.2, 3.3.3 and 3.3.4.

3.3.2 Tensile Static and Dynamic Tests

The indirect tensile test (Adedilimila, 1980) is used to determine tensile modulus, compressive modulus, Poisson's ratio and tensile strength, under static conditions of loading. The indirect tensile test can be used to evaluate the above mentioned parameters for bitumen stabilized (bituminous mixtures), cement stabilized and lime stabilized pavement materials. The test provides data that reproduces the load – deformation characteristics, that approximates the plane strain behaviour realized in pavements by wheel loads. These characteristics may be obtained for both the vertical and horizontal orientation of specimen material tested.

Some tests are also able to determine tensile moduli of deformation under dynamic conditions. Such a test is the dynamic (repeated flexural) stiffness test developed by Deacon (1965). The test has its use in determining the tensile dynamic modulus (stiffness) of bituminous mixtures.

It takes into account the effects of fatigue (repeated flexure) on the modulus measured. Schmidt (1972) developed the diametral resilient modulus test, which measures the dynamic tensile modulus of bituminous mixtures. It is similar to the indirect tensile test but it has a provision for repetitive loading. Therefore, it incorporates the fatigue effects on modulus measured.

3.3.3 Static Compressive Tests and the Asphalt Mix Stiffness Procedure

The fundamental compressive tests under static conditions of loading include the triaxial compression shear test by Bishop and Hankel (1957), and the asphalt mix stiffness procedure developed by Bonnaure et al (1977). In addition, the asphalt mix stiffness procedure has provisions for determining dynamic modulus (stiffness) of bitumen binders and hence their bituminous mixtures.

The triaxial compression shear test (Bishop and Hankel, 1957) is a compressive type of test. It determines the internal stability and the stress-strain characteristics of all pavement materials under both vertical and lateral pressures. Internal stability is determined in the form of shear strength. From stress-strain characteristics analyses, compressive modulus of deformation can be determined. In the test, there is provision for excess pore water pressure measurements. The test can also be equipped with a device to apply loading to specimens being tested. As such the test can be used to determine the static and dynamic values of shear strength and compressive modulus of deformation. Visually, all pavement materials viz: bituminous bound, cement bound, aggregates inclusive of granular subgrade soils and cohesive subgrade soils (Pell and Brown, 1972) can be analyzed by this test.

In a conventional triaxial compression test, a cylindrical sample is loaded axially to failure, at a constant confining pressure. Conceptually, the peak value of the axial stress is taken as the confined compressive strength of the sample. In view of the variability of sample properties, when adequate samples are available, repeated testing is carried out to determine average values. For the purpose of this study, the peak value of the axial stress is referred to as the maximum stress.

The confining pressures are very important in that they provide a lateral support which prevents the sample or the pavement structure for that matter from spreading laterally.

Confinement also helps to minimize the vertical displacement. In practice, confinement of the pavement structures is attained by use of kerbs and shoulders. Confinement is also initially attained by a way of compaction whether by use of rollers during construction or by traffic when a pavement is already in use.

Previously, several researchers have used the triaxial compression test to characterize asphalt mixtures for permanent deformation. The researchers include Rabbira (2002), Kamal et al (2005), Muraya (2007) and Habtamu (2008). In all the studies, the investigations were done in the laboratory and compared with results from the field tests. The studies found a good comparison of the results obtained from the laboratory and field tests.

The asphalt mix stiffness procedure, Bonnaure et al (1977) has two provisions. The first provision is a nomographic solution (Heukelom, 1973), to obtain the stiffness of modulus of bitumen. The second provision uses Equation 3.1 (Heukelom, 1973), to convert this value to the stiffness modulus of bituminous mixture in question. The Equations 3.1, 3.2 and 3.3 are used to take into account the effect of volume composition of aggregates, air voids and bitumen in the mixture, respectively.

Different research workers developed a procedure for predicting the stiffness modulus of bituminous mixtures through three stages. In the first stage, it was Van der Poel (1955), who showed that the stiffness modulus of bituminous mixtures (S_m) depended on the stiffness modulus of bitumen (S_b) and the volume concentration of aggregates (C_v).

Where;

$$C_v = \frac{V_g}{V_g + V_b}$$

V_g = Volume of aggregates

V_b = Volume of bitumen in the mixture.

The second stage was reached by the extension made on the Van der Poel (1955) work by Heukelom and Klomp (1964). They suggested the following equations.

$$S_m = S_b \left(1 + \frac{2.5}{n} \frac{C_v}{1-Cv}\right)^n \quad \ldots\ldots 3.1$$

Where; $n = 0.831 \log_{10} \frac{4.0 \times 10^{10}}{S_b}$

S_m = Stiffness modulus of bituminous mixtures

S_b = Stiffness modulus of bitumen

V_g = Volume percentage filled by aggregates in the bituminous mixture.

V_b = Volume percentage filled by bitumen in the bituminous mixture.

(S_m and S_b in N/m^2)

Equation 3.1 was developed using results from bituminous mixtures containing around 3% air voids (V_a). Where V_a is the volume percentage filled by air in the bituminous mixture. Since the air voids percentage (V_a) has a significant influence on the stiffness modulus of bituminous mixtures, Equation 3.1 is not suitable for mixtures with higher air void contents. A minimum percentage of air voids is required to give the bituminous mixtures capacity to elastically deform without rutting (Noureldin et al, 1994).

Van Draat and Sommer (1965) cleared this shortcoming in the third stage. They proposed that a correction to the volume of concentration of aggregates (C_v) for air void contents in excess of 3% should be used. The proposal culminated into Equation 3.2;

$$C'_v = \frac{100 * C_v}{100 + V_a - 3} \quad \ldots\ldots \quad 3.2$$

Where; C'_v = corrected volume concentration of aggregates.

$$C_v = \frac{V_g}{V_g + V_b}$$

C_v = Volume concentration of aggregates in the bituminous mixture.

V_g = Volume percentage filled by aggregates in the bituminous mixture.

V_b = Volume percentage filled by bitumen in the bituminous mixture.

V_a = Volume percentage filled by air in the bituminous mixture.

Therefore the final equation for predicting the stiffness modulus of bituminous mixture is;

$$S_m = S_b \left(1 + \frac{2.5}{n} \frac{C'_v}{1-C'v}\right)^n \quad \ldots\ldots \quad 3.3$$

Where;

$$n = 0.831 \log_{10} \frac{4.0 \times 10^1}{S_b}$$

S_m and S_b in N/m^2, C'_v = Corrected volume concentration of aggregates as given in equation 3.3.

Therefore, using Equations 3.1, 3.2 and 3.3 depending on the air voids content (V_a) in a bituminous mixture, the static and dynamic compressive modulus of bituminous mixtures at various loading times (or frequencies) and temperatures can be determined. The compressive modulus of deformation at given conditions of loading time (t) (frequency) and temperature (T) was defined by Van der Poel (1954) as stiffness [$S_{(t, T)}$]. This was to differentiate it from the modulus (E) of elastic responses. Static values of stiffness are taken as those determined at frequencies less than 1Hz (or loading times of greater than 1 second). On the other hand, dynamic values of stiffness are those determined at frequencies greater than 1Hz (or loading less than 1 second).

Creep test determines a quantity called creep compliance under compressive static conditions. This quantity expresses the relationship between stress, strain and time of viscoelastic pavement materials under static loading. Creep compliance can be determined for such materials as bituminous mixtures and used as input in viscoelastic pavement analysis and design models.

3.3.4 Poisson's Ratio of General Pavement Materials

The tests available for determination of Poisson's ratio in the laboratory and in the field consist of static and dynamic tests. In both types of tests, deformations are monitored in both lateral and axial directions. Generally, in the static laboratory tests, volume change of the test specimen may be measured or the lateral and axial deformations monitored. When volume change is measured the appropriate equation for the evaluation of Poisson's ratio is given by Equation 3.4 (Yoder and Witczak, 1975).

$$\mu = \tfrac{1}{2}\left(1 - \frac{1}{\varepsilon_a}\frac{\Delta V}{V_0}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots .. \; 3.4$$

Where;

ΔV = Change in volume of specimen

V_0 = Original volume of specimen

ε_a = Axial strain measured in direction in which loading is applied.

For most pavement materials, the influence of many factors on Poisson’s ratio for a given material is normally small (Yoder and Witczak, 1975). This fact allows the use of typical assumed values of Poisson’s ratio, for different pavement materials, for use in pavement analysis and design instead of direct testing. However, Poisson’s ratio of bituminous mixtures is very sensitive to temperature while that of cohesive subgrade is sensitive to moisture content.

In the event of measuring lateral and axial deformations, Equation 3.5 is used to estimate Poisson’s ratio value.

$$\mu = - \varepsilon_x/\varepsilon_y = - \varepsilon_z/\varepsilon_y \quad \text{.......} 3.5$$

Deformation can be converted to strain by dividing it by the original dimensions of the material specimen expressed in the same units as deformation. The dynamic test type of Poisson’s ratio involves wave propagation testing. This test is appropriate for the determination of Poisson’s ratio insitu. The governing Equation 3.5 above is presented in Equation 3.6 below.

$$\mu = \frac{\beta - 1}{\beta - 2} \quad \text{.......} 3.6$$

Where;

$\beta = 2\,(V_s/V_p)^2$

V_s = Shear velocity obtained from vibration testing.

V_p = Compression wave velocity evaluated from seismic refraction testing.

Table 3.1 shows Poisson’s ratio values used by various pavement design agencies.

Table 3.1; Poisson's Ratio Values Used by Various Pavement Design Agencies

Materials	Original Shell Oil Company	Revised Shell Oil Company	Test Asphalt Institute	Kentucky Highway
Asphalt Concrete	0.5	0.35	0.40	0.40
Granular Base	0.5	0.35	0.45	0.45
Subgrade	0.5	0.35	0.45	0.45

Source; Kaliti (2007)

Chapter four

Methodology and Data Collection

4.1 Introduction

The simplest way to characterize the behavior of a flexible pavement under wheel loads is to consider it as a homogenous half-space. A half – space has infinitely large area and an infinite depth with a top plane on which the loads are applied. Permanent deformation is an important factor in flexible pavement design. To estimate the rut depth, it is necessary to determine the permanent deformation parameters of the material for each layer.

In pavement engineering, structural pavement design goes hand in hand with the materials design. The structural analysis, design and construction of a pavement are essentially meant to build into the structure provisions to guard against possible distress and subsequent failures. The pavement constituent material properties used in the structural analysis and design of the pavement are estimated in laboratories using laboratory designed materials. Hence, the methods used to characterize pavement materials should simulate adequately the actual type of loadings in service.

Pell and Brown (1972) gave four categories of pavement materials as reported by Kaliti (2007). These include bituminous – bound, cement-bound, unbound granular materials inclusive of cohesionless soils and cohesive subgrade soils. Further, with the increase in traffic load and tyre pressure, most of the permanent deformation occurs in the upper layers rather than in the subgrade (Huang, 2004).

Hence, out of the four categories, bituminous mixtures have been chosen for investigation in this study program. As described in the literature review, rutting has been found to be affected by contact stress distribution, pavement structure, mixture composition, gradation, type of aggregate surface texture, filler, voids in asphalt mixture and compaction among others. These are the variables which can be varied in order to be able to evaluate their contribution towards the resistance of permanent deformation by asphalt layers in a pavement.

In order to develop appropriate laboratory testing methods so as to obtain material properties associated with a given distress, it is necessary to ensure that the laboratory method chosen simulates adequately the actual field loading response which is associated with the same distress. There are two categories of pavement distress namely; structural distress and functional distress (Kaliti, 2007).

The major structural distress of concern in the design of bituminous pavement include fatigue cracking, rutting, shearing, crushing and permanent deformation in a bituminous layer. On the other hand the functional distresses that are mainly as a result of poor materials design include disintegration, fracture and distortion.

Huang (2004) recognizes that the permanent deformation of paving materials depends strongly on the testing methods and procedures used for preparing and testing the specimens. These variations together with the uncertainty in traffic and environmental conditions make the prediction of rut depth or permanent deformation extremely difficult. Therefore the use of simplified methods is warranted.

The simplified methods are only applicable if equivalent single-axle load and an average set of material properties are used throughout the design period. It is not possible to apply this method, because the stresses in each layer can be any values, depending on the axle loads and the environmental condition throughout the year. A more practical approach is to perform a large number of permanent deformation tests under various stress and environment conditions and develop regression equations relating permanent strains to these conditions. These regression equations can then be incorporated into a multilayer computer program to compute the rut depth as illustrated by Allen and Deen (1986) and reported by Huang (2004).

A well constructed pavement has a kerb at the sides and layers of surfacing, wearing course, base, subbase and subgrade below so that only the upper side is exposed to the traffic loading and environmental factors such as moisture and temperature. As a result, any failure of the asphalt pavement occurs when there is internal instability in the bituminous bound material making the pavement.

A laboratory test which is able to simulate failure of asphalt mixtures as a result of internal instability of the material will be more suitable for characterizing asphalt mixtures for permanent deformation. Hence triaxial compression test is considered to be more suitable.

Figure 4.1 shows an overview of test program used in this research. The initial stage involved the design of the different asphalt mixtures considered in this research. The purpose of the mixture design was to determine the composition of the total asphalt mixture and to facilitate the determination of the composition of the aggregate skeleton and bitumen /mortar components. Apart from enabling the composition of the different components, the mixture design also served in the provision of the target densities for the total asphalt mixture and aggregate skeleton test specimens. The mixture design was followed by the development of test specimen preparation protocols. The specimen preparation protocols consisted of development of the specimen preparation techniques for the total asphalt mixture; bitumen and aggregate skeleton. Prior to the development of the aggregate skeleton preparation protocols, the aggregate skeleton for the different asphalt mixtures were identified. The total asphalt mixture was characterized by use of triaxial compression test in the next phase of this research.

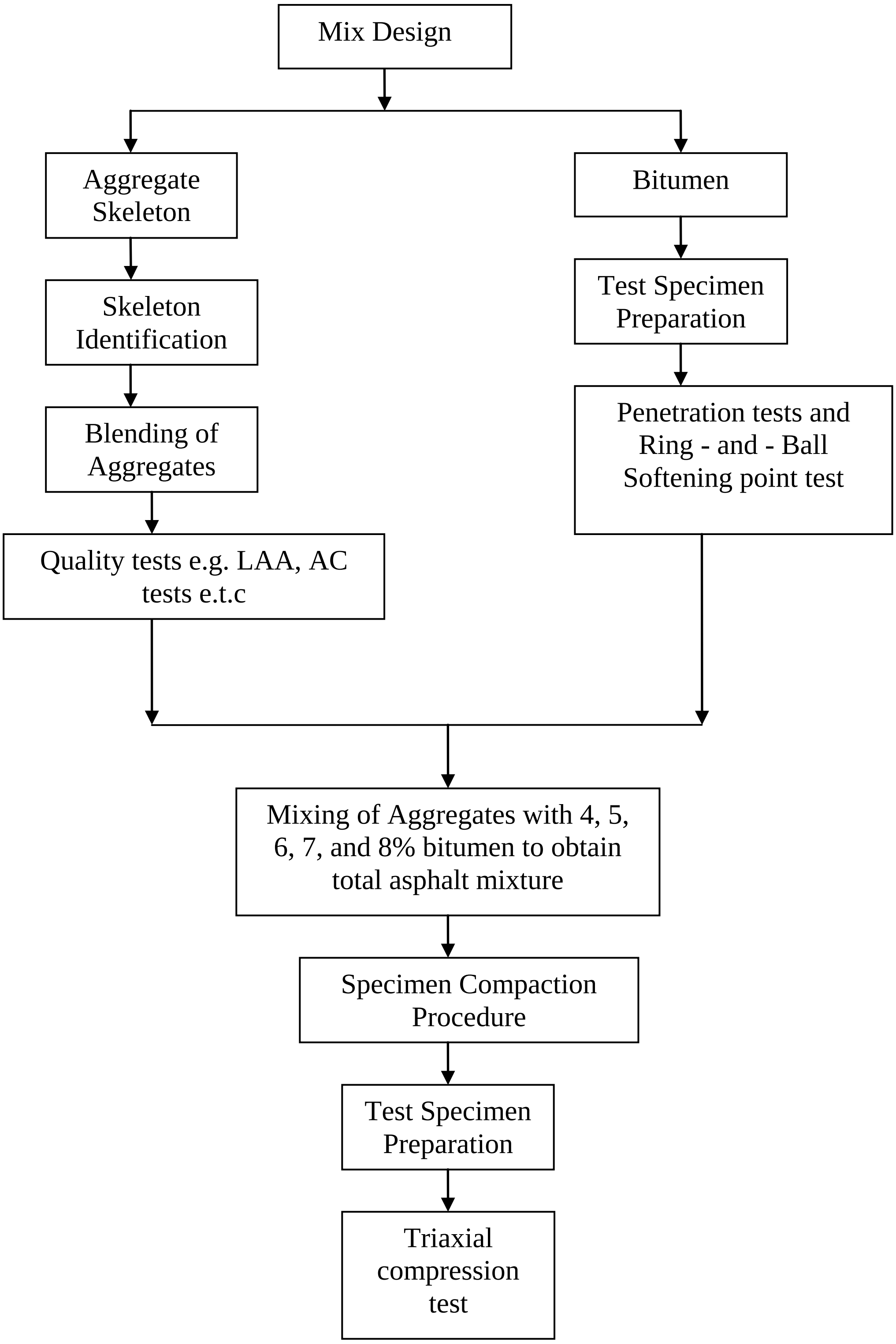


Figure 4.1; Overview of test Program

4.2 Permanent Deformation Analysis

Fig 4.2 gives an overview of permanent deformation analysis. The tests on the total mixture will provide information on the elasticity characteristics of the mixture.

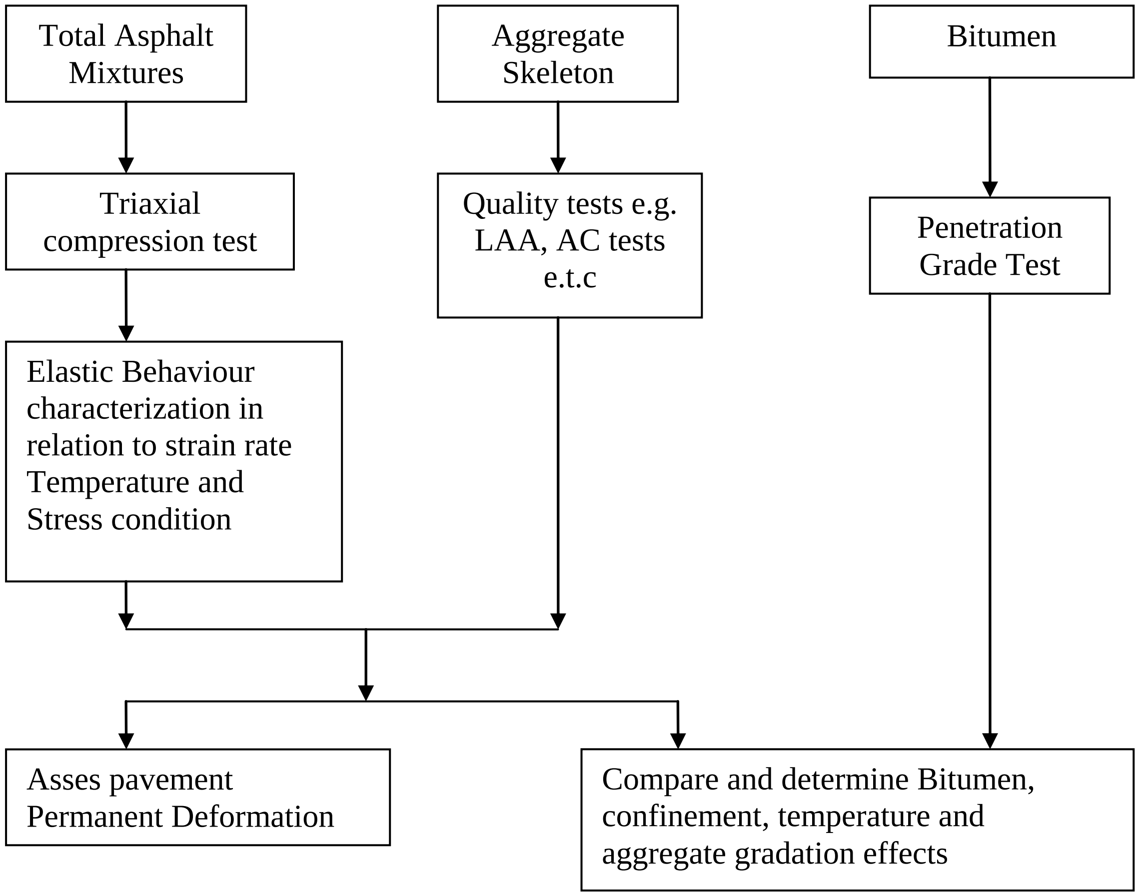


Figure 4.2; Overview of permanent deformation analyses

4.3 Materials Design

4.3.1 Aggregates

Kandhal and Mallick (1999) showed that dense aggregate gradations are desirable to mitigate the effects of rutting. When properly compacted, mixtures with dense or continuous aggregate gradations have fewer voids and more contact points between particles than open or gap-graded mixtures. Pell and Brown (1972) concluded that a gap – graded mixture exhibits more deformation than a continuously graded mixture. They argued that this is due to less aggregate interlock in the gap – graded mixture. They further argued that, because aggregate interlock becomes more important at high temperatures, gap – graded mixtures may be even more susceptible to rutting at higher temperatures.

The aggregates used in this study program had nearly the same geological properties as those used in pavement construction in Nairobi and its environs. The pavements around Nairobi represent a good case of highly loaded road due to its subjection to high volume of vehicular traffic class T1. The traffic in roads neighboring the city centre is found to move at slow speeds due to traffic jam representing a case of nearly static loading condition on the pavements. According to a study by Kaliti (2007), the aggregates used in pavement construction around Nairobi City are usually obtained by crushing the Nairobi Phonolite rock.

The sample aggregates were acquired from Kay Construction Limited Quarry at Mlolongo and transported to the University of Nairobi Highways Laboratory. The aggregates were graded, tested and blended based on specifications before the mixture design was carried out. The processes were carried out to model a pavement design and construction and in accordance with the Ministry of Roads Specifications of Kenya. The aggregates obtained from site were generally classified into three categories namely; fine, coarse and filler aggregates. The classification was necessary in order to simplify the application of the laid down procedures in the Road Design Manual part III by the Ministry of Roads and Public Works, (1987). According to the manual, coarse aggregates are the ones which are retained on 2.4mm sieve Number 7BS. The aggregates which pass 2.4mm sieve and are retained on 0.075mm sieve Number 200BS are called fine aggregates. The filler aggregates are defined as the ones passing 0.075mm sieve. The process of sieve analysis was carried out on representative samples in order to obtain the three aggregate categories.

The Road Designers Manual and the Standard Specifications of Road construction in Kenya dictate that the aggregates to be used for pavement construction should be of high quality. In order to ensure that the aggregates used in this study met the quality requirements, various tests were carried out which included water absorption test, specific gravity test, aggregate crushing value test, aggregate impact value test, Los Angeles Abrasion Test and Shape tests. The shape tests gave information like Elongation Index, Flakiness Index and Angularity Number. For the fine aggregates, the only tests carried out were absorption test and Specific Gravity Test. Specific gravity test was the only test carried out on filler made of cement. The test procedures for the tests mentioned above were in accordance to the British Standard Institution Manual (1975). Tests on the surface roughness and texture were not necessary as the aggregates were freshly obtained from the site.

The aggregates acquired fell into three individual nominal sizes namely; 10/14 mm, 6/10mm and 0/6mm. The quantity of each individual nominal size was 150kg. 100kg of Portland cement was also acquired and utilized as filler. The grading of the Portland cement was acquired from the manufacturer (East Africa Portland Cement Company). The actual grading of the aggregates was accomplished through sieving each of the three individual nominal sizes of the aggregates. Figures 4.3 and 4.4 show the particle size distribution of the Portland cement and the individual aggregates respectively.

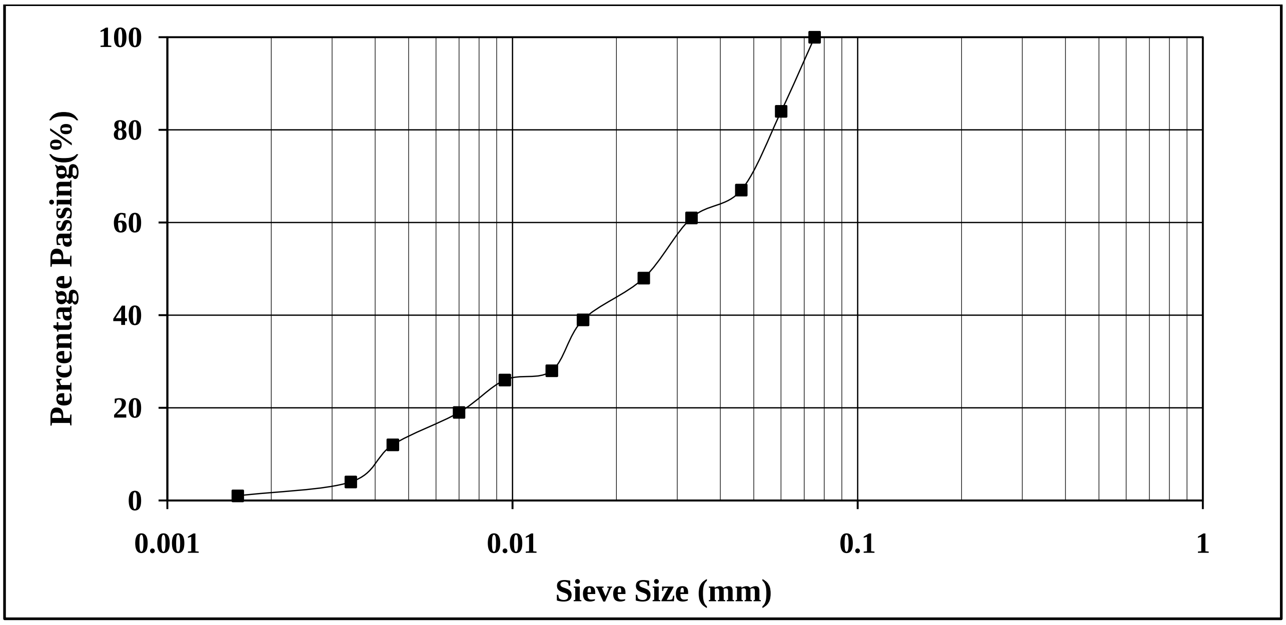


Figure 4.3; Particle Size Distribution of Portland cement

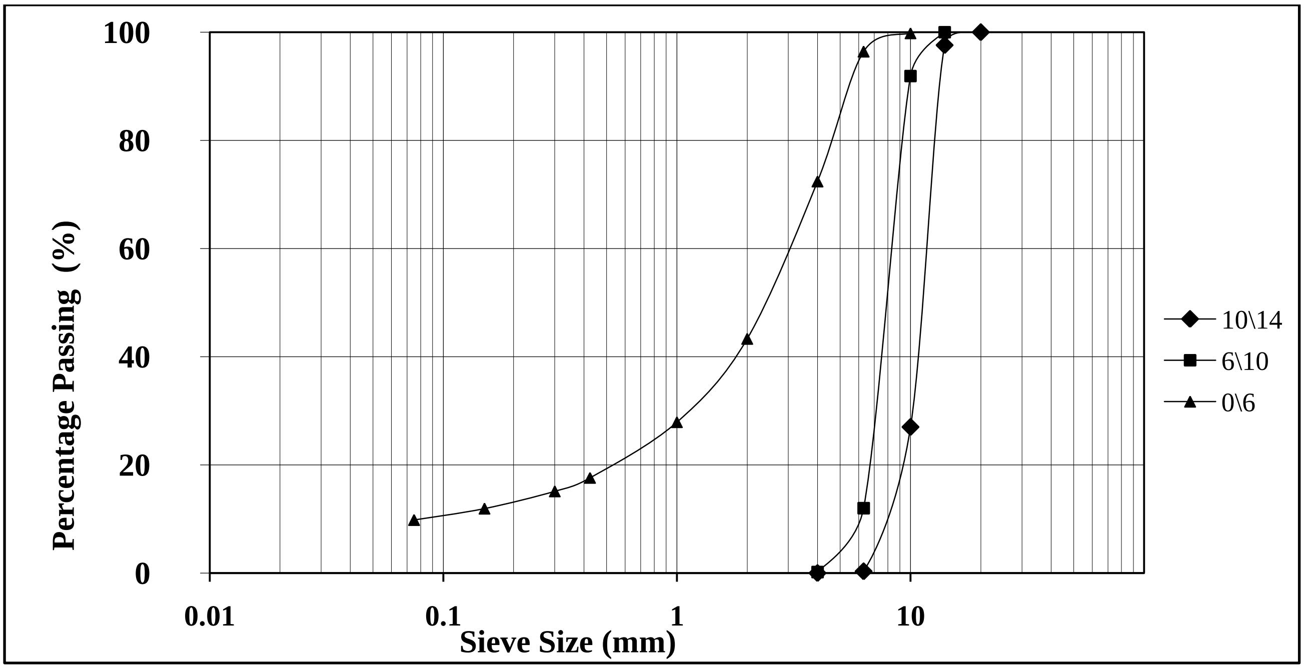


Figure 4.4; Particle Size Distributions of the Individual Aggregate stocks

Table 4.1 shows the results of quality tests carried out on the three categories of aggregates (coarse, fine and fillers). The results obtained and the corresponding values used as criteria as given by the standard specifications of Roads Department (1987) and Road Research Laboratory (1962), to qualify their acceptance for use in the design of bituminous mixtures were used in the study. As explained later, different gradations were chosen to form pavement skeletons for asphalt mixtures to be investigated.

Table 4.1: Empirical Test Results on Aggregates and Acceptance Criteria Values

Test	Classification of Aggregate Material					
	Test Results			Acceptance Criteria Value		
	Coarse > 2.4mm	Fine < 2.4mm	Filler < 0.075mm	Coarse > 2.4mm	Fine < 2.4mm	Filler < 0.075mm
Aggregate Crushing Value(A.C.V)	16	-		<10 - >35 (v.s-v.w) Max 23	-	-
Aggregate Impact Value(A.I.V)	15	-	-	<10 - >35 (v.s-v.w) Max 20	-	-
Los Angeles Abrasion Test (L.A.A)	20	-	-	Max 25	-	-
Water Absorption Test	3%	2.5%	-	0. 1 - 4%	0. 1 – 0.4%	-
Specific Gravity	2.67	2.65	3.0	2.5-3.0	2.5-3.0	2.5-3.0
Elongation Index	16	-	-	Max 25	-	-
Flakiness Index	9	-	-	Max 25	-	-
Angularity Number	10	-	-	About 10	-	-

Note: Values of Portland cement obtained from East Africa Portland Cement Company

v.s = Very Strong and v.w = Very Weak

Specifications of mineral fillers for bituminous paving mixtures are covered by ASTM D 242 Standard. The standard recommends that Portland cement shall be sufficiently dry to flow freely and essentially free from agglomerations. It is a requirement of the standard that 100% of the filler should pass through sieve No.30 (600μm) and at least 70% of it should pass through sieve No.200 (75μm).The standard requires that the filler be free of organic impurities and have plasticity index not greater than 4.However the standards (ASTM D 242) exempts fillers made of ordinary Portland cement from the latter requirement.

4.3.2 Penetration Grade Bitumen

Mahboub and Little (1988), using uniaxial creep testing found that less viscous asphalts make the mixtures less stiff and therefore more susceptible to irrecoverable deformations i.e. rutting. Monismith, et al (1985) made similar observations and recommended harder (more viscous) asphalt cements in thicker pavements and hotter climates.

Penetration grade bitumens are usually used in the construction of heavy duty flexible pavement in Kenya. The choice of penetration grade is based on the environmental factors, more so temperatures of the road surface at the locality of the pavement and expected traffic. Commonly used bitumen in Kenya is in penetration range of 40-200, (Kaliti, 2007). For example for heavily trafficked road at high temperatures, a low penetration grade say 60/70 will be more suitable. For lightly trafficked road and at low temperatures, high penetration grade bitumen say 180/200 will be more suitable. In order to investigate the effect of penetration grade of bitumen on permanent deformation of asphalt mixtures, three penetration grades were investigated which included 60/70, 80/100, and 180/200 penetration grade bitumen.

The bitumen was acquired in six containers each having a capacity of 20 litres. Each of the three penetration grade was held in two containers making up a total of 120 litres of bitumen. Each of the six containers containing bitumen was heated on an electrically powered hot plate and three samples were extracted from each container. Extractions were done carefully from a small hole at the top of the containers. The holes were then sealed to avoid contamination by foreign materials.

It is a requirement that a test be carried out on bitumen samples in order to investigate their consistency properties, the effect of temperature on their consistencies and their specific gravities. The samples extracted as explained above were subjected to these tests. The consistency was investigated by carrying out the penetration test in accordance with ASTM D5 Standards. The effect of temperature on consistency of bitumen sample was determined by ring -and-ball softening point test (ASTM D2398) and finally, the specific gravity was determined by the specific gravity of the bitumen test (ASTM D90). Table 4.2 shows the average results of consistency and specific gravity tests.

Table 4. 2; Average Test Results on Penetration Grade Bitumen

Test	Grade of Penetration Bitumen		
	60/70	80/100	180/200
Penetration (P)	65	90	190
Softening Point (SP)	50	48	42
Specific Gravity (SG)	1.04	1.06	1.06

4.3.3 Bituminous Mixture Design

4.3.3.1 Introduction

In order to ensure the homogeneity of the prepared bitumen samples, it was necessary to ensure that the basic ingredients of the mixture came from the same source. This was important so that the prepared samples were as similar as possible to prevent unnecessary variation in the test results. It was also ensured that the design procedure was the one used in designing of our Kenyan roads more so the roads in Nairobi and its surroundings. The sample preparation procedures e.g. mixing methods and compaction were similar to the ones used or specified by Kenyan or adapted standards. This was important so that the mixture was a representative of Kenyan roads. Similarly the specified time for curing before the pavement can be opened for traffic, was adopted and the samples were allowed to cure for seven days before the tests could be carried out.

After meeting the latter requirements, it can be argued that realistic test specimens which reasonably duplicated the corresponding insitu paving asphalt in all its major aspects including composition, density and engineering properties were produced. The test specimens can be said to have facilitated laboratory study of the bituminous mixtures performance with respect to roads distress

The asphalt mixtures used in this study were generally obtained in two stages. The first stage involved mix design with the second stage being that one of preparation of test specimens. First the gradation of aggregates was done and then followed by mix design to determine the optimum bitumen content.

4.3.3.2 Mix Design

The Marshall or Hveem method is generally selected as preliminary design tool in the determination of adequate asphalt content. Monismith et al, (1985) recommended that the mixture have an asphalt content such that the air – void content would be approximately 4 percent. To minimize the instability problems and therefore permanent deformation, they recommended an absolute minimum of 3 percent air voids. These criteria are necessarily associated with mixtures of adequate stability resulting from the use of high quality aggregates. Mahboub and Little (1988) indicated that larger asphalt contents producing lower air voids increased rutting potential. They suggested that the reduction in air voids as a result of increased asphalt content indicates that the void space is becoming filled with asphalt. As a result, the increase in asphalt content is equivalent to the introduction of lubricants between aggregate particles otherwise separated by a very tight network of air voids. This phenomenon causes the mixture with the higher asphalt content to be more susceptible to permanent deformation.

Muraya (2007) found that good resistance to permanent deformation requires low voids in the mineral aggregate (VMA) and that the desirable grading for minimum VMA can be determined using dry aggregate tests. However he cautioned the lowest theoretical VMA could be undesirable as it may not allow sufficient voids in the aggregate for enough binder to ensure satisfactory compaction without the mixture becoming overfilled.

Reducing air voids up to a point increases the resistance of the mixture to rutting. In the field, a low air – void content is generally achieved with higher compactive energy. Uge and Van de Loo (1974) found that the relative displacement of mineral particles occurring when an asphalt mixture is handled at high temperature (during laying and compaction) or at moderate temperature but under prolonged loading are of the same nature. Therefore, to minimize the rutting effect, they recommended the use of harsh mixtures; those of comparatively poor workability; and heavy rollers. Such a combination should result in an improved arrangement of the mineral skeleton and thereby an increase in internal friction. They concluded that harsh mixtures, thoroughly compacted after laying, are very resistant to permanent deformation.

Compaction is also a critical factor in preparing specimens for laboratory evaluation. The purpose of any laboratory compaction process is to simulate, as closely as possible, actual compaction produced in the field. Factors such as the orientation and interlocking of aggregate particles, the extent of inter particle contact, air – void content and void structure and number of interconnected voids should be closely reproduced.

To prepare representative laboratory specimens for permanent - deformation testing, “shearing” deformations are essential to the compaction process. Moreover, they demonstrate the importance of measuring the permanent – deformation characteristics under conditions representative of those occurring in the field. Hence the densification expected in the mixture due to repeated trafficking must be properly duplicated in the laboratory.

During the mix design, first the individual gradings of the aggregates were blended to achieve the desired gradation while ensuring that it complied with the specified gradation envelope. Then, the optimum bitumen content to bind with the blended aggregates was established. The binding of the aggregates with bitumen resulted into a mixture with the required engineering properties and quality. The target bitumen mix in this case was an asphalt concrete type 1 which is appropriate for use as a binder or wearing course for heavily trafficked roads (Traffic class T1) due to its ability to resist permanent deformation. In this regard, the aggregate gradation envelope used was as in Table S2b of MOR&PW Design manual part iii (1987).

Four blended aggregates were made from the Kenya’s Ministry of Roads specifications (1987) and labeled 1, 2, 3, and 4. Table 4.3 shows the individual nominal aggregate proportions in both percentage and weights for each of the four blended aggregate grading. Between 1 and 2% of Portland cement was used as a filler as shown in table 4.3. Then, the blended aggregates were analyzed and the results used to come up with the gradation curves. The gradation curves in comparison to the Ministry of Roads and Public Works envelopes for the four gradations are as shown in the Figures 4.5, 4.6, 4.7 and 4.8. The MOR&PW envelopes dictate that the designed aggregates skeleton gradation must lie within it.

Aggregate blend 1 was gap – graded and hence it resulted into a gap – graded asphalt mixtures. Blend 2, 3 and 4 were well graded and hence they resulted into continuously graded asphalt mixtures. This way the researcher was able to investigate the effects of aggregate gradation on permanent deformation of asphalt mixtures.

Table 4.3: Percentage and Weight of Each Fraction in a Marshall Specimen of 1100g

Nominal size Aggregate	PERCENTAGE AND WEIGHT OF FRACTION USED							
	BLENDED AGGREGATE GRADATION							
	1		2		3		4	
	%	Wt.(g)	%	Wt.(g)	%	Wt.(g)	%	Wt.(g)
0/6	81	891	78	858	75	825	75	825
6/10	2	22	4	44	7	77	12	132
10/14	16	176	16	176	16	176	11	121
Filler (cement)	1	11	2	22	2	22	2	22
Total	100	1100	100	1100	100	1100	100	1100

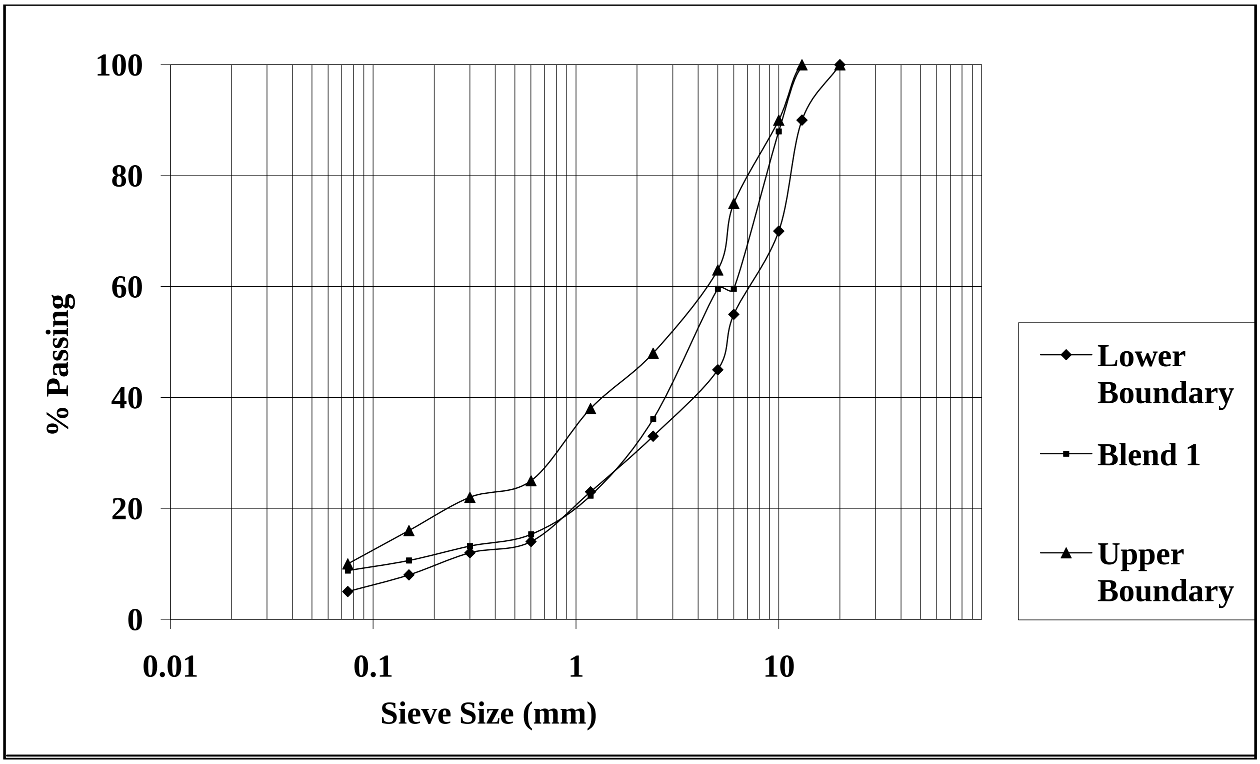


Figure 4.5; Aggregate Blend 1 Gradation in Comparison With MoRPW Envelope

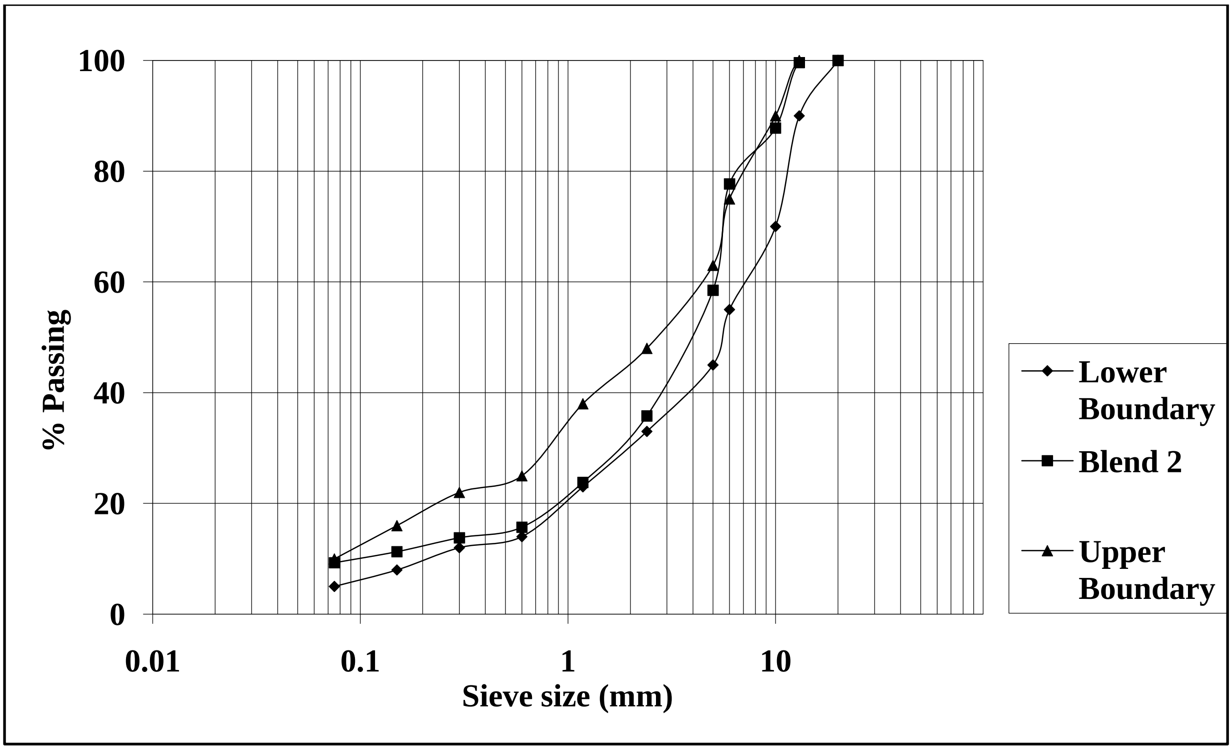


Figure 4.6; Aggregate Blend 2 Gradation in Comparison With MoRPW Envelope

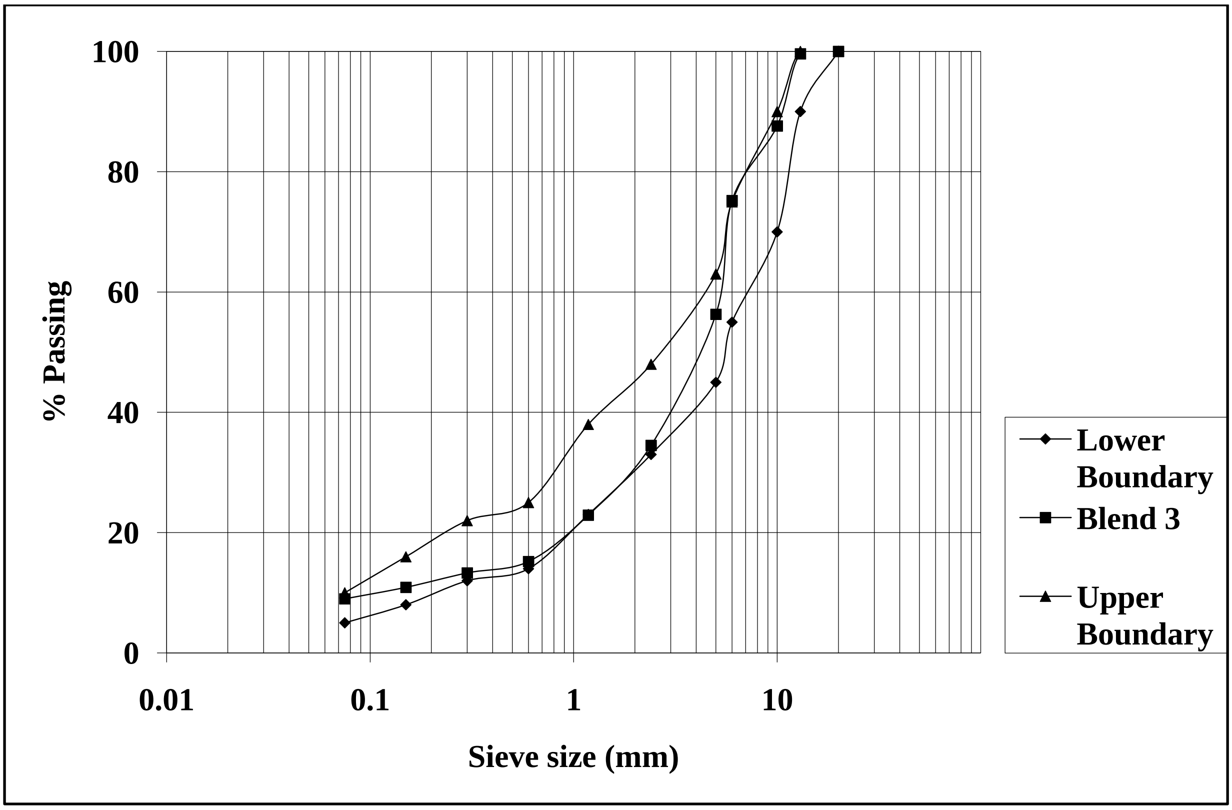


Figure 4.7; Aggregate Blend 3 Gradation in Comparison With MoRPW Envelope

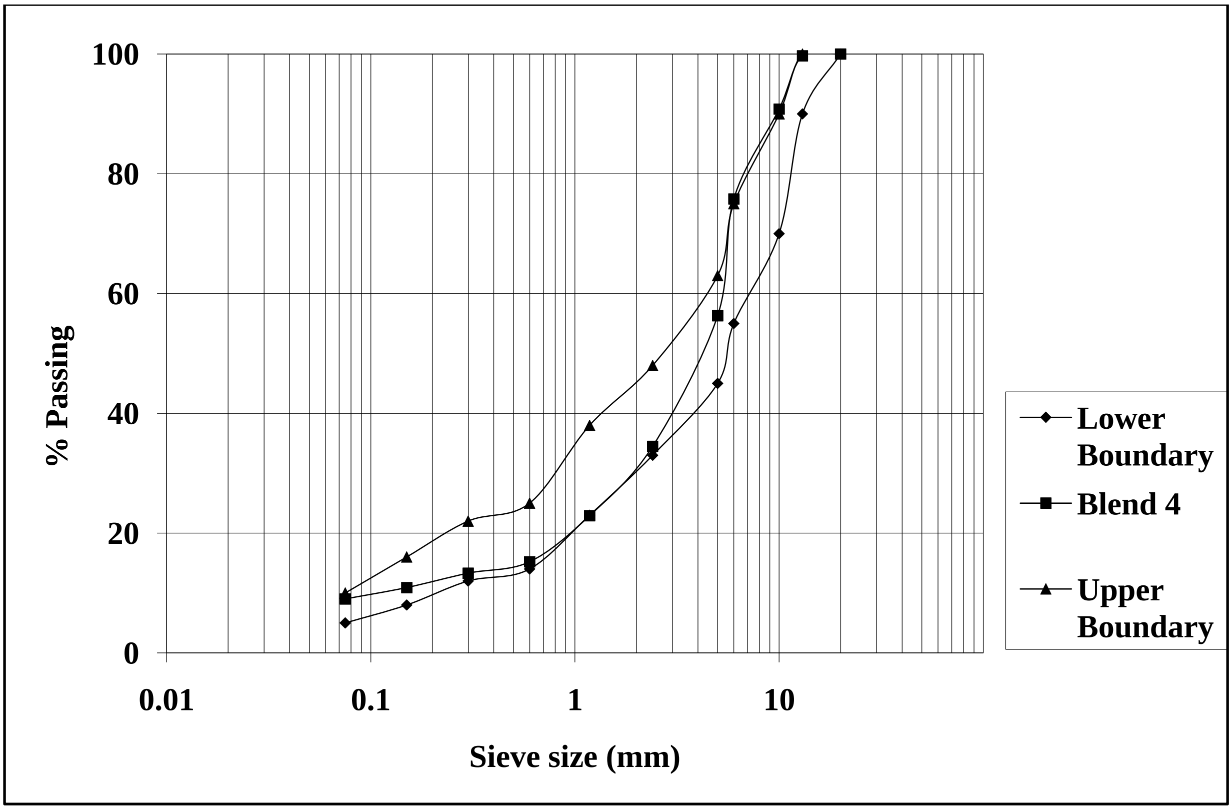


Figure 4.8; Aggregate Blend 4 Gradation in Comparison With MoRPW Envelope

Cement filler was used with its gradation below 0.075mm (0.1mm on a log scale) sieve size as shown in Figure 4.3. It is expected that this will reflect on the blended aggregate gradation curves Figure 4.5 to 4.8. However this is not the case because the Kenyan Roads Design Manual allows for envelope of only up to 0.075mm (0.1mm on log scale) and any particle sizes which passes through 0.075mm sieve size are recognized as a single percentage shown as the curve extent between 0.01 and 0.1 mm in log scale in Figure 4.5 to 4.8.

Marshall Method of mix design (ASTM D1559) was adopted in establishing the optimum bitumen content for each of the four blended aggregate gradations. As Kaliti, (2007) puts it, the main reasons behind the use of the method are; (i) it is the only method used in Kenya to design mixtures in the laboratory, and (ii) it has gained a wide acceptance world wide as compared with other methods like Hveem method, Hubbard - field method, Job mix method and Smith Triaxial method. The adoption of the method ensured that the bituminous mixtures used in this study were designed in the same way as those used in the construction of Kenyan Roads.

In the mix design, the weight of every Marshall specimen used was 1100g. For each blended aggregate grading, the right proportion by weight of the individual nominal aggregates that compose a 1100g Marshall specimen were calculated as shown in Table 4.3. The calculated weights of dried individual nominal aggregates were weighed on a balance. The weighed proportions were mixed and put together in a paper bag and clearly labeled with the blended aggregate gradation 1, 2, 3 and 4.

For the purpose of mix design, there were 45 Marshall specimens for each blended aggregates type 1, 2, 3 and 4 broken down as; 3x5x3 where 3 is the different penetration grade bitumens (60/70, 80/100 and 180/200), 5 is the various bitumen contents (4, 5, 6, 7 and 8%) and finally the last 3 is for triplicate samples whose results were averaged to minimize errors. Hence in total, for the four blended aggregate gradations, 180 (45x4) Marshall Specimens were prepared.

The specimens were kept in an atmosphere of free air in laboratory to cure for at least seven days and subsequently tested using the Marshall Test apparatus.

Before testing the Marshall specimens, each specimen had its average dimensions of height and diameter and weight measured and recorded against its table of readings. A pair of Vernier calipers was used to measure the height and the diameter. Then the Marshall specimens were separately weighed both in air and water. The measurements read on the electronic balance were recorded.

After the above measurements, the specimens were kept in hot water bath whose temperatures were maintained by a thermostat at 60 ± 1^{0}C. The specimens were kept in hot water for 30-40 minutes after which they were expected to have attained the temperature of the hot water (i.e. 60 ± 1^{0}C). The Marshall Apparatus specimen breaking head was also kept in hot water bath and removed only when testing the specimen so as to avoid the specimen losing heat to the cold breaking head during testing period. Then, the Marshall specimens were recovered from the hot water bath and put between well lubricated lower and the upper jaws of the Marshall breaking head and loaded. The Marshall machine used was digital and during the testing it advanced the breaking head against a specimen at a rate of 50.8mm or 2 inches per minute. Hence the Marshall testing represented a case of controlled strain loading.

The Marshall testing of the mix design specimens produced two results which included the Marshall Stability and Marshall Flow. The Marshall stability is the maximum load in pounds or kilograms that a specimen can take before failure. The Marshall flow is the vertical deformation at failure of the specimen which translates into the displacement of the upper jaw breaking head relative to the lower jaw at the time of specimen failure. All the Marshall specimen results were entered against its label in the tables specifically prepared for the results. From the above physical measurements, the raw mix design data was obtained. The design mix data was processed in accordance with the Marshall Mix design procedure (ASTM D1559). All the processed results from the physical measurements are as shown in Tables 1.1 to 1.12 in appendix 1. The results are for the four blended aggregate gradations 1,2,3 and 4 and three penetration grade of bitumen 60/70,80/100 and 180/200. The following abbreviations have been used in the above mentioned tables;

CDM – Compacted Density of the Mix
SGM – Specific Gravity of the Mix
VMA – Voids in the Mixed Aggregates
VIM – Voids in the Compacted Mix
VFB – Voids Filled With Bitumen

Their values were estimated using equations 1.1 to 1.11 in appendix 1. Then, the mix design graphs were plotted using processed results on each of the Tables 1.1 to 1.12 in appendix 1. Graphs from each table were plotted from the processed results described above. The graphs plotted from the analyzed Marshall test results given in each of the twelve tables were seven in number from each table as shown in the group of Figures 1.1.1 to 1.12.7 in Appendix 1.

The first mix design graph i.e. Figure 1.1.1, appendix 1, was obtained by plotting unit weight of the mix (lb/ft^3) (being equal to CDM (g/cm^3) multiplied by 62.4 against percentage bitumen content. The second graph (i.e. Figure 1.1.2 appendix 1 presents Marshall Flow value in units of 0.01 inch against percentage bitumen content. The third graph i.e. Figures 1.1.3 Appendix 1 gives the percentage voids in the compacted mix (%VIM) against percentage bitumen content. The fourth graph i.e. Figure 1.1.4 Appendix 1 shows the behaviour of the percentage aggregate voids filled with bitumen (%VFB) with percentage bitumen content. The fifth graphs i.e. Figure 1.1.5 Appendix 1 gives Marshall bearing capacity (Marshall Stiffness) in lb/0.01 inch against the percentage bitumen content. The sixth graph Figure 1.1.6 presents the Marshall stability (lb) against the percentage bitumen content. Lastly the seventh graphs i.e. Figure 1.1.7 appendix 1 gives the percentage voids in the mixed aggregate (%VMA) against the percentage bitumen content.

The graphs in the Figure groups of 1.1.1 to 1.1.7, 1.2.1 to 1.2.7, 1.3.1 to 1.3.7 presents the mix design data for mixes prepared using the blended aggregates gradation coded 1 and bitumen penetration grades 60/70, 80/100 and 180/200 respectively. The graphs in Figure groups of 1.4.1 to 1.4.7, 1.5.1 to 1.5.7 and 1.6.1 to 1.6.7 presents the mix design data for mixes obtained by mixing the blended aggregate gradation coded 2 and bitumen penetration grade 60/70, 80/100 and 180/200 respectively. The graphs in Figure groups of 1.7.1 to 1.7.7, 1.8.1 to 1.8.7 and 1.9.1 to 1.9.7 presents the mix design data for mixes obtained by mixing the blended aggregate gradation coded 3 and bitumen penetration grade 60/70, 80/100 and 180/200 respectively. Finally, the graphs in Figure groups of 1.10.1 to 1.10.7, 1.11.1 to 1.11.7 and 1.12.1 to 1.12.7 presents the mix design data for mixes obtained by mixing the blended aggregate gradation coded 4 and bitumen penetration grade 60/70, 80/100 and 180/200 respectively. Two steps were followed in the design of the mixes. The first stage involved obtaining a first approximation of the design optimum bitumen content which was obtained by finding from the mixed design graphs the percentage bitumen contents that corresponded with maximum stability, maximum unit weight (C.M x 62.4), 5% voids in the mix (VIM) and 75% voids filled with bitumen (VFB). An average of these four percentages of bitumen content was computed and taken as the first approximated design optimum bitumen content. (FADO).The second step consisted of using the first approximated design optimum bitumen content obtained in the first design step, to obtain the actual design optimum bitumen content (ADO). This was done by a way of iteration. Several values of the bitumen content close to the first approximated design optimum bitumen content was taken. The corresponding mix design Marshall Quantities of each chosen percentage of bitumen content were read of from the mix design graphs. These were recorded on a table of readings and compared with contemporary mix design criteria values as given in mix design manuals. The actual design optimum bitumen content percentage was the one that satisfied most of the design criteria.

Tables 4.4(a) and 4.4(b) show the first approximated design optimum bitumen content (FADO) and the actual design optimum bitumen content (ADO), respectively .The corresponding

Marshall mix design results for each of the two bitumen contents i.e. (FADO) and (ADO) as read off from the mix design graphs are compared with the mix design criteria values applied in design of all the mixes (Jackson and Brien, 1962 and Roads Department, 1987). Tables 4.4(a) and 4.4(b) gives the said comparisons for every mix designed with a blended aggregate gradation and penetration grade bitumen.

For each of the four blended aggregate gradations and the three penetration grades of the bitumen used to design mixes from the four gradations, Actual Design Optimum percentage bitumen content was obtained. This was the optimum bitumen content used in the preparation of the designed specimens for testing by triaxial compression test that comprised the epicenter of this research.

Table 4.4(a): Design Mix Parameters for First Approximated Design Optimum Bitumen Content (F.A.D.O)

SERIAL NO.	MARSHALL MIX PROPERTY	BLENDED AGGREGATE GRADATION												RECOMMENDED DESIGN CRITERIA VALUE RANGE (Asphalt Institute,1994)
		1			2			3			4			
		PENETRATION GRADE OF BITUMEN												
		60/70	80/100	180/200	60/70	80/100	180/200	60/70	80/100	180/200	60/70	80/100	180/200	
1	% BITUMEN CONTENT	5.8	5.8	6.1	5.5	6.6	6.0	6.1	6.1	7.0	5.6	5.6	5.2	MIN 5.0 MAX 8.0
2	STABILITY (Kg/0.254mm)	850	835	743	1276	851	987	726	816	635	891	1021	883	MIN 227Kg,0.3MpaT.P MIN 318Kg,0.5MpaT.P MIN 454Kg,0.7Mpa T.P
3	% VOIDS IN TOTAL MIX(V.I.M)	4.3	4.2	5.1	2.5	6.1	5.5	5.5	6.0	6.0	4.6	4.6	4.1	3±1 to 8±1 for roadbase 3±1 to 5±1 for surfacing
4	% AGGREGATE VOIDS FILLED WITH BITUMEN (V.F.B)	75	75	70.9	84.6	69.3	72.5	71.6	67.5	72.3	71.3	70	70	MIN.75±5 % MAX 85±% FOR ROADBASE MIN.55±5 % MAX 75±% FOR SURFACING
5	% VOIDS IN MIXED AGGREGATES (V.M.A)	14.9	16.4	17.6	15.7	17.5	15.8	19.2	17.6	20.0	16.2	16.4	15.2	MIN. 16% MAX.19%
6	COMPACTED DENSITY OF THE MIX (C.D.M)	2.117	2.115	2.123	2.115	2.111	2.109	2.158	2.116	2.130	2.165	2.067	2.173	SPREAD AREA PER TONNE OF BITUMINOUS MIXTURE (M^2/TONNE)
7	MARSHALL STIFFNESS (Kg/0.254mm.)	38	37	46	59	44	42	55	35	36	33	43	49	>1.2 x T.P
8	FLOW	15.2	18	16.3	18	20	22	13	17	16	17.5	21	20	MIN 8 MAX 18

Table 4.4(b): Design Mix Parameters for Actual Design Optimum Bitumen Content (A.D.O)

SERIAL NO.	MARSHALL MIX PROPERTY	BLENDED AGGREGATE GRADATION												RECOMMENDED DESIGN CRITERIA VALUE RANGE (Asphalt Institute,1994)
		1			2			3			4			
		PENETRATION GRADE OF BITUMEN												
		60/70	80/100	180/200	60/70	80/100	180/200	60/70	80/100	180/200	60/70	80/100	180/200	
1	% BITUMEN CONTENT	5.1	5.2	5.5	5.3	5.5	6.1	5.5	6.0	6.1	5.8	5.9	6.2	MIN 5.0 MAX8.0
2	STABILITY (Kg/0.254mm)	841	816	726	1275	956	907	726	791	680	794	907	810	MIN 227Kg,0.3MpaT.P MIN 318Kg,0.5MpaT.P MIN 454Kg,0.7Mpa T.P
3	% VOIDS IN TOTAL MIX(V.I.M)	4.6	4.9	6.3	2.5	6.4	6.9	5.5	6.5	6.6	5.1	5.2	4.3	3±1 to 8±1 for road base 3±1 to 5±1 for surfacing
4	% AGGREGATE VOIDS FILLED WITH BITUMEN (V.F.B)	74	69	62.5	84.6	70.9	69	71.6	63.5	62.3	66.9	67.3	69	MIN.75±5 % MAX 85±5% FOR ROADBASE MIN.55±5 % MAX 75±5% FOR SURFACING
5	% VOIDS IN MIXED AGGREGATES (V.M.A)	15	16.1	16.8	15.7	17.1	16.2	19.2	17.3	19.8	16.2	16	15.2	MIN. 16% MAX.19%
6	COMPACTED DENSITY OF THE MIX (C.D.M)	2.117	2.115	2.123	2.115	2.111	2.109	2.158	2.116	2.130	2.165	2.067	2.173	SPREAD AREA PER TONNE OF BITUMINOUS MIXTURE (M^2/TONNE)
7	MARSHALL STIFFNESS (Kg/0.254mm.)	43	45	50	59	55	50	55	41	53	45	54	57	>1.2 x T.P
8	FLOW	15	18	14.5	18	18	18	13	17	13	16	18	18	MIN 8 MAX 18

Plates 4.1 to 4.4 show the researcher at various stages of the study.

Plate 4.1; The researcher loading aggregates into polythene bags at Kay Construction Quarry in Mlolongo, Athi river

Plate 4.2; The researcher unloading aggregates in polythene bags for storage at the University of Nairobi Highways Laboratory

Plate 4.3; The researcher preparing specimens for testing at the University of Nairobi Highways Laboratory

Plate 4.4; A sample being prepared for crushing using Marshall Apparatus during mix design

4.4 Materials Characterization for Rutting

4.4.1 Introduction

As stated in chapter one, it is the aim of this research to characterize asphalt mixtures for permanent deformation using triaxial compression test method. The characterization of asphalt mixtures is achieved through several objectives which dwell on investigating how various factors both volumetric and environmental affect the permanent deformation of asphalt mixtures. In a situation where the pavements are made by use of dense bitumen macadam as a base and asphalt concrete as a binder or wearing course, most often the asphalt concrete wearing course will deform by a way of rutting as it gets squeezed over more stiff bitumen macadam. The rutting is found to occur in form of vertical displacement due to densification and lateral displacement due to shear failure. The latter has been attributed to be a major cause of permanent deformation in asphalt pavements.

The Kenyan Ministry of Roads design manual part III (1987), recommends that the pavements which are designed to carry traffic class T1 be made of a wearing course of asphalt concrete type 1. This type of asphalt concrete is made of continuously graded aggregates which provide a perfectly interlocked mix bound by straight run bitumen to provide a homogenous mix with considerable stiffness to resist permanent deformation. The wearing course is the one which is mostly affected by the permanent deformation (rutting) whenever the pavements are loaded. Further, most of the deformations in bituminous pavements are found to occur on the inclined lanes used by the slow moving vehicles. This considerable rutting can be associated with the increase in longitudinal and lateral contact stresses. The increase in these stresses results from the increase in the component of the wheel load acting on the inclined surface. This is so because static loadings results in high strains as compared to the dynamic loadings. It is worth noting that fast moving vehicles results in a dynamic loading while the slow moving ones result in to a static loading.

The aggregate gradation envelope for asphalt concrete type 1 wearing course is 0/14. As such the asphalt concrete prepared for characterization by triaxial compression test had its aggregate skeletons within this envelope.

According to Muraya (2007), asphalt mixtures share many similarities with soil mixtures. The first scenario is that of continuously graded asphalt mixtures where the stone skeleton held together by the mortar plays an important role in offering resistance to permanent deformation. Initially the friction forces generated in the contact points carry the loads applied to the skeleton and skeleton will only show a limited amount of deformation. If however the load becomes too high, deformations will occur and because the skeleton is so well compacted it will not show a decrease in volume but an increase as a result of dilation. When dilation occurs, the mortar is subjected to tension hence underlining the importance of adhesive characteristics of the mortar to the aggregate and the tensile characteristics of the mortar. The second scenario is that of gap graded mixtures where; if the void content is low, pore pressures as observed in saturated soils might develop resulting in instability in the mixture. Furthermore, the tensile characteristics of the mortar and adhesive characteristics of the mortar aggregate interface become important if dilation occurs. In such a mixture, the large aggregate stones do not really form a skeleton but more or less float in the mortar. The large aggregate particles do not play a role in providing resistance to permanent deformation. This resistance must be provided by the mortar thus emphasizing the importance of the visco-elastic properties in such a mixture. The type of bitumen, type of filler and the amount of bitumen and filler mainly determine these properties. If the void content is too low, pore pressure effects as observed in saturated soils may occur resulting in instability of the mixture.

The preceding discussion show that asphalt mixtures share many similarities with soil mixtures and it also demonstrates that the skeleton and the mortar play specific roles. The discussion also underscores the importance of adhesive and tensile characteristics of the mortar. Therefore, use of triaxial test to characterize the asphalt mixtures is justified.

4.4.2 Triaxial Testing

4.4.2.1 Introduction

Triaxial testing offers a possible means of testing the asphalt mixtures at different confinement levels. The stress invariants from this test offers a possible means of linking pavement stresses to stress condition that can be generated in the laboratory under triaxial test conditions.

4.4.2.2 Test Set-up

The test set-up for the asphalt mixtures as used in this research consisted of the digital ELE Tritest 50 triaxial cell and frame shown in Plate 4.5. Also available was the old manual triaxial machine as shown in Plate 4.5. ELE Tritest 50 machine is capable of applying a maximum vertical load of 50kN, with the loading platens capable of moving at a speed between 0.00001 and 9.9999mm/min. The triaxial cell of the old manual triaxial machine comprised of a vertical hydraulic actuator and a pneumatic water confinement generation system capable of delivering a maximum of about 15kN and 800kpa respectively. The cell pressure was applied by a means of an air bellow that pressurized the water to the required confinement level. The triaxial cell was capable of performing permanent deformation test conducted under a constant cell pressure. The triaxial machine had two dial gauges where the axial deformation (strain) and the load causing the deformation were read.

4.4.3 Preparation of Asphalt Mixtures for Characterization by Triaxial test

The asphalt mixtures were prepared using Marshall Procedure as discussed in Section 4.3. The only difference as compared to Marshall Sample is that the triaxial sample had smaller dimensions and hence smaller weight. The dimensions of the cylindrical triaxial samples used in this research were 38mm diameter and 76mm height. Assuming a target maximum dry density (MDD) of 2100kg/m^3 at 5% optimum moisture content (OMC) and 95% compaction, the weight of one triaxial sample was computed to be 200g.

In the mix design, the weight of every triaxial specimen used was 200g. For each blended aggregate grading , the right proportion by weight of the individual nominal aggregates that compose a 200g Triaxial specimen were calculated as shown in Table 4.5. The calculated weights of dried individual nominal aggregates were weighed on a balance. The weighed proportions were mixed and put together in a paper bag marked corresponding to the blended aggregate gradations 1, 2, 3 and 4.

Plate 4.5; Digital triaxial test machine (upper photo) and the manual triaxial test machine (lower photo) at the University of Nairobi soils laboratory.

Table 4.5: Percentage and Weight of Each Fraction in a Triaxial Specimen of 200g

Nominal size Aggregate	PERCENTAGE AND WEIGHT OF FRACTION USED							
	BLENDED AGGREGATE GRADATION							
	1		2		3		4	
	%	Wt.(g)	%	Wt.(g)	%	Wt.(g)	%	Wt.(g)
0/6	81	162	78	156	75	150	75	150
6/10	2	4	4	8	7	14	12	24
10/14	16	32	16	32	16	32	11	22
Filler (cement)	1	2	2	4	2	4	2	4
Total	100	200	100	200	100	200	100	200

Objective number one of this research was achieved by preparing four types of aggregate blends labeled 1, 2, 3 and 4. The aggregates complied with the MOR&PW design manual part iii chart S2b. Specific aggregate sizes were varied to obtain four different particle size distribution curves as shown by Figures 4.5 to 4.8. This enabled the evaluation of the effect of gradation on permanent deformation of asphalt mixtures. Objective number two was achieved by using three different types of penetration grade bitumen (60/70, 80/100,180/200) as a binder to the prepared aggregate blends 1, 2, 3 and 4. Objective number three was achieved by varying the bitumen content of the prepared asphalt mixtures from 4% to 8%, so that each of the four blends of aggregates were bound by 4, 5, 6, 7 and 8% of each of the three penetration grade bitumen. Objective number four was achieved by conducting the triaxial test for each of the prepared sample at five different loading temperatures namely 20, 30, 40, 50 and 60^{0}C likely to be encountered on Kenyan roads. To control the temperature a thermometer was fitted in the triaxial cell. Objective number five was achieved by carrying out the triaxial tests for all the prepared asphalt mixtures at three distinct levels of confining stresses namely 0.1Mpa, 0.2Mpa and 0.4Mpa. Objective number six was achieved by using a power law and natural logarithm relationships to correlate the various measured parameters of the triaxial test.

For the purpose of triaxial test, there were 675 specimens for each blended aggregate types 1, 2, 3 and 4 broken down as; 3x5x5x3x3 where 3 is the different penetration grade bitumens (60/70, 80/100 and 180/200), 5 is the various bitumen contents (4, 5, 6, 7 and 8%), 5 is the five different samples for each bitumen content for testing at five different temperatures (20, 30, 40, 50 and 60^{0}C), 3 is the three different samples for each bitumen content for testing at three levels of confinement (0.1, 0.2 and 0.4Mpa) and finally 3 is the number per sample whose results were averaged to minimize errors. Hence in total, for the four blended aggregate gradations 2700 (675x4) triaxial Specimens were prepared as shown in Table 4.5(a). Further, additional 180 samples were prepared using the optimum bitumen content (as shown by Table 4.4[b]) obtained during the Marshall Mix procedure. The 180 samples were also tested using triaxial method (180 = 4x3x5x3 where 4 is number of blended aggregates, 3 is the number of penetration grade bitumens, 5 is the number of loading temperature and 3 is the number per sample averaged to reduce errors).

Table 4.5(a): Number of Prepared Triaxial Test Specimens

Blend	Number of Samples Prepared					Total
	Consistency (60/70,80/100,180/200)	Bitumen content (4, 5, 6, 7, 8%)	Confinement (0.1, 0.2, 0.4Mpa)	Temperature (20, 30,40, 50, 60^0C)	No. per sample	
1	3	5	3	5	3	675
2	3	5	3	5	3	675
3	3	5	3	5	3	675
4	3	5	3	5	3	675
					Total	2700

The specimens were kept in an atmosphere of free air in laboratory to cure for at least seven days and tested as from the eighth day using Triaxial Test apparatus. The seven days curing period corresponds to the duration of seven days that the newly laid bituminous layers in new asphalt flexible pavements in Kenya are supposed to cure before the pavements are opened to traffic (Roads Department, 1987).

Before testing the triaxial specimens each specimen had its average dimensions of height and diameter and weight measured and recorded against its table of readings. A pair of Vernier calipers was used to measure the height and the diameter. Then the triaxial specimens were separately weighed both in air and water. The measurements read on the electronic balance were recorded. The results are as tabulated in Appendix 2 Tables 2.1 to 2.3.

Temperature has been found to have a significant effect on rutting. Kamal et al (2005) determined from their study that rutting increased by a factor of 250 to 350 with a temperature increase from 20^{o}C to 60 ^{o}C. Linden and Van der Heide (1987) reported a significant increase in rutting in Europe during the very hot summers of 1975 and 1976.The major challenge in this study was how to test asphalt mixtures at different loading temperatures given the fact that the confinement inside the triaxial cell was to be provided by water supplied by use of a pipe which was connected to a cold water source on one side. First the sample plus the loading platens were put in water bath with temperatures corresponding to the loading temperatures. The side of the pipe connected to the water source was unplugged and water which was initially heated to the corresponding loading temperatures was fed until the triaxial cell was full. Then a considerable amount of motor vehicle lubricating oil was fed into the pipe before connecting this pipe to the cold water source. The oil was critical in separating the heated water from the cold tap water while at the same time offering continuity of the flow needed to pressurize the triaxial cell in order to achieve the desired confinement. The temperature at the triaxial cell was regulated by means of a thermometer.

The triaxial compression test provided data which made it possible to plot vertical stress versus axial strain curves. From these curves, five major properties which are critical in characterizing the asphalt mixtures for permanent deformation were obtained. These properties included;

- Maximum stress
- Tangent stiffness
- Stress at initiation of plasticity
- Poisson's ratio
- Stress at initiation of dilation

Plates 4.6 to 4.9 show the researcher at various stages of the laboratory works.

Plate 4.6; Prepared asphalt concrete sample ready for testing by use of triaxial machine

Plate 4.7; Mixing bituminous materials for preparation of asphalt mixture samples to be tested using triaxial compression test

Plate 4.8; The researcher weighing the various nominal aggregate sizes for blending

Plate 4.9; The researcher holding the already weighed aggregates ready for blending

Chapter five

Data Analysis and Discussion

5.1 Introduction

In this chapter two main aspects are to be handled. The first is the discussion of how the data obtained from research methodology and model was analyzed. The second is the discussion of the data analysis results in comparison with previous work.

All the results obtained were put in an organized manner with a view of obtaining the implications of the factors being investigated. The results were analyzed and discussed with the sole purpose of achieving the objectives set in chapter one.

The data which are considered in this chapter fall into four main categories namely;

- Aggregates data
- Penetration grade bitumen data
- Marshall mix design data
- Compression triaxial test data

The various engineering parameters used to characterize asphalt mixtures for permanent deformation in the study were obtained by use of a triaxial compression test. To obtain the triaxial test samples, Marshall Mix design procedures were used, the main reason being that they are the same procedures used in Kenya for design of bituminous pavement structures. To be able to characterize the asphalt mixtures for permanent deformation, various parameters and testing conditions were varied in order to get a picture of their effects in asphalt pavements, especially for the top layers that are wearing course and binder course. These factors were aggregate gradation, bitumen consistency, loading temperature, bitumen content and confining stresses as stated in the objectives in chapter one.

To investigate the effects of aggregates on permanent deformation, four blends of aggregates namely 1, 2, 3 and 4 were prepared. To obtain these four blends of aggregates, the grading was varied by use of different percentages of particle sizes. This way, well graded aggregates or gap graded aggregates were obtained. This is critical as it gives an insight into what type of aggregates gradation is able to resist permanent deformation at different conditions such as high loading temperatures or at different levels of bitumen content.

To evaluate the effects of bitumen content on the pavement deformation of asphalt mixtures, each of the four blends of aggregates mentioned above, was mixed with 4%, 5%, 6%, 7% and 8% of bitumen, respectively. This is important as it allows for investigation into the effect of bitumen content towards resistance of permanent deformation of asphalt mixtures. One is able to know how a well graded or gap graded skeleton behaves at different bitumen contents. As such the designers are able to know how to design pavements which have more resistance of permanent deformation especially at high loading temperatures, by varying the aggregate gradation as well as the bitumen content. There was also a second set of asphalt mixtures prepared with the bitumen content this time round being determined by Marshall Mix design procedures.

The bitumen used to bind the aggregate skeletons prepared as mentioned in the latter paragraph is likely to influence the behaviour of asphalt mixtures towards resistance of permanent deformation especially when its consistency at various temperatures is taken into consideration. To investigate this factor, three types of penetration grade bitumens were used in the study to prepare various asphalt mix specimens for testing by use of compression triaxial test. The three types of penetration grade bitumen used were 60/70, 80/100, 180/200.

The penetration grade 60/70 bitumen has got the highest consistency at high temperatures and the asphalt mixtures made of this type of bitumen are expected to be less susceptible to the high temperature effects. On the other hand, if the loading temperatures are too low, asphalt mixtures made using 60/70 penetration grade bitumen will be highly susceptible to permanent deformation inform of cracking.

180/200 penetration grade bitumen has got the highest inconsistency at high temperatures and the asphalt mixtures prepared using this type of bitumen are expected to be very susceptible to the effects of high loading temperatures but they perform well when the loading temperatures are low. 80/100 penetration grade bitumen has got moderate consistency at high temperatures and the asphalt mixtures made using this type of bitumen are expected to exhibit moderate susceptibility at high or low loading temperatures. The loading temperature effects on the permanent deformation of asphalt mixtures was evaluated by testing all the prepared specimens at five different temperatures expected in Kenyan roads namely 20, 30, 40, 50 and 60^0C. This way a highway engineer is able to tell which gradation of aggregate skeleton, at what bitumen content and consistency is able to resist permanent deformation and at what temperature is able to do so. This is achieved by comparing test results of similar asphalt mixtures tested at different loading temperatures.

All the prepared asphalt samples were tested at three levels of confining stresses (0.1Mpa, 0.2Mpa and 0.4Mpa) apart from being tested at different loading temperatures. In real pavements, confinement is provided for by introducing kerbs or shoulders on the sides of the asphalt pavement lanes. Confinement ensures that the main pavement is contained hence reducing the lateral stresses. Two sets of results were obtained from triaxial compression test. First were the results of asphalt mixtures prepared using aggregates blend 1, 2, 3 and 4 and bound by 4,5,6,7 and 8 percent bitumen content. Secondly, the results of tests on asphalt mixtures from aggregates blend 1, 2, 3 and 4 bound by optimum bitumen content as determined by Marshall Mix design procedure as explained in chapter 4.

It is worth noting that the Marshall Mix design procedure was only used for the purposes of designing the mixtures and not for the real characterization of asphalt mixtures for permanent deformation. Characterization of asphalt mixture for permanent deformation was done by use of triaxial compression test data.

5.2 Material Test Data

5.2.1 Introduction

Two types of materials were used in the study which included the crushed mineral aggregates and the penetration grade bitumen. In practice, during the pavement construction, the qualifying tests are performed on each material depending on the types of construction, traffic loading and environmental conditions expected in the field. The tests are normally empirical aimed at indicating the ability of the materials to resist the prevalent in service conditions. The results of the quality test are compared with the corresponding acceptance criteria values as given in various standards such as our own Roads Designer's Manual part III (1987) and other various British or American standards.

5.2.2 Mineral Aggregates Test Data

Aggregate skeleton constitute a very major component in asphalt mixtures. They contribute towards the strength of asphalt mixtures. For aggregate skeletons, what matters is the composition of each and every particle sizes. The main contributing factors of crushed mineral aggregates to overall engineering properties of asphalt mixtures as pavement material include their type, grading and basic qualities. The types of aggregates used in this study were crushed from Nairobi Phonolite rock and hence for all purposes they could be considered to be of the same mineral content. The grading of the aggregates considered included that of individual nominal aggregates sizes 14mm, 10mm, 6mm and 3mm and that of blended aggregate gradations 1, 2, 3 and 4. The quality of aggregates was determined using the existing technological or empirical available methods such as the ones recommended by Road Designer's Manual part III (1987) and various British and American standards.

5.2.2.1 Aggregates Grading

The coefficients of uniformity (C_u) and of concavity or curvature (C_z) were used to evaluate the aggregates grading as defined by ASTM D2487. A large value of the coefficient of uniformity (C_u) corresponds to a large range in aggregate sizes and this indicates well-graded aggregates. A value of coefficient of uniformity of 1 represents aggregates that are of the same size and therefore uniform. In practice, aggregates with a coefficient of uniformity lying in the range of 1 to 4 are considered uniform. A coefficient of curvature C_z of about 1 to 3 implies that the aggregates are well graded. The values of coefficient of curvature that are much less than 1 or that are much greater than 3 imply that aggregates are poorly graded.

Equation 5.1 and 5.2 (Cernica, 1982) were used for the estimation of the coefficients of uniformity (C_u) and curvature (C_z) respectively;

$$C_u = \frac{D_{60}}{D_{10}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; 5.1$$

$$C_z = \frac{D_{30}^2}{D_{10}D_{60}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; 5.2$$

In the two equations 5.1 and 5.2, D_{10} is the biggest size of the aggregate particles in the smallest 10% finer by weight on the particle size distribution curve. D_{30} is the biggest size of the aggregate particles in the smallest 30 percent finer by weight on the particle size distribution curve. D_{60} is the biggest size of the aggregate particles in the smallest 60% finer by weight on the particle size distribution curve. The analysis of the gradations of the four blended aggregates 1, 2, 3 and 4 reveals that they are well graded except gradation 1, which by all means appears to be falling towards being gap-graded. The conclusion is informed by the evidence of grading curves shown from figures 4.5 to 4.8 and the calculated coefficient of uniformity and curvature as shown in table 5.1. Looking closely to figure 4.5, the curve is found to have a horizontal part indicating some missing aggregate sizes and hence gap graded aggregate gradation.

Table 5.1; Values of coefficient of Uniformity (C_u) and curvature (C_z) of blended gradation.

Blended Aggregate Gradation Code	Particle diameter, D at 10%, 30 and 60% finer (mm)			Coefficient of uniformity $C_u = \frac{D_{60}}{D_{10}}$	Coefficient of curvatures $C_z = \frac{D_{30}^2}{D_{10}D_{60}}$	Deduction
	D_{10}	D_{30}	D_{60}			
1	0.14	1.4	5	35.7	2.8	Well-graded
2	0.09	1.3	5.3	58.9	3.5	Well graded
3	0.1	1.5	5.5	55	4.1	Well graded
4	0.1	1.6	5.5	55	4.7	Well graded

Key

- A value of coefficient of uniformity, $C_u > 4$ corresponds to a well graded material.
- A coefficient of curvature C_z of about 1 to 3 implies that the materials are well graded.

5.2.2.2 Mechanical Strength and Quality Tests on Aggregates.

Two things were taken into consideration when conducting the mechanical strength and quality test of aggregates. One factor is the ability to transfer loading by the aggregate skeleton through the bituminous mixture structures. The second range of factors considered are the nature, intensity, speed, frequency of traffic loading and prevailing environmental conditions such as temperature and drainage conditions.

In chapter four of this study, it was mentioned that the mixtures designed for the test were equivalent to those ones used on a busy road with slow moving traffic around Nairobi. The general expectation is that the traffic loading is nearly static and of high intensity and frequency. The cumulative number of standard axles, ESAL, over design period lies in class T_1 of standard axles 2.5×10^7 to 6.0×10^7 as per MOR&PW Designer Manual part III (1987). The pavement temperature is in the range of 20^0C to 50^0C, Gichaga (1979). This means that the materials used as aggregate skeleton must be of high strength and quality.

Table 4.1 shows the technological tests on the aggregates, their results and acceptance criteria values. The acceptance criteria values were obtained from the Roads Design Manual part III (1987), Road Research Laboratory (1962) and Jackson and Brien (1962). The values consist of the ranges of mechanical strengths, qualities and specific gravity indices which have proved satisfactory in pavement construction through long-term construction experience. The mechanical strength indices consist of Aggregate Crushing Value (ACV), Aggregate Impact Value (AIV) and Los Angeles Abrasion (LAA) test results. The quality indices comprise of water absorption test results, elongation index, angularity number and flakiness index. The specific gravity values (SG) of both the coarse aggregates and fine aggregates were necessary for the proportioning and compatibility of the aggregates skeleton in gradation.

From Table 4.1, the coarse aggregates mechanical values are way above the set criteria which is an indication that they qualify for use in pavement construction. Such aggregates are able to withstand compaction by use of a hammer in laboratory or a roller during the real construction without disintegrating. Also they are expected to withstand the traffic loading without having to crush. The water absorption test results and those of the specific gravity lie within the values set by the criteria. The shape test results are above the set minimum values which are an indication that the coarse aggregates have got good internal friction. The fine aggregates were investigated by use of water absorption test and the specific gravity tests with the results found to be within the range given by the acceptance criteria as in Table 4.1.

The adhesion of aggregates to the binder in a bituminous mix is enhanced by the aggregates porosity. This is because the binder is able to easily penetrate into the aggregates particles. When aggregates are not coated with bitumen, their degree of porosity is an indicator of how easily they breakup under traffic loading especially after weakening upon water absorption or adsorption.

The specific gravity values of the filler (Portland cement), was obtained from the manufacturer (East Africa Portland cement company, Athi River). The specific gravity values of fine and coarse aggregates are very close to each other, Table 4.1. The closeness of the specific gravities is healthy for it ensures good balancing of the aggregates in a given gradation.

The value of coefficient of uniformity (C_u) and curvature (C_z) as tabulated in Table 5.1 shows that all the blended aggregates, 1, 2, 3 and 4 are well graded. However, having a closer look into the grading curves in Figures 4.5 – 4.8, it can be seen that Figure 4.5 has a horizontal section. This horizontal section shows that sieve 2mm and sieve 1mm are passing the same percentage by weight of aggregates. This means that the aggregate sizes between these two sieves are missing in this gradation blend 1. Hence, the blended aggregate 1 can be termed as gap – graded and within the MoR&PW grading envelope as shown in table S2b of MoR&PW design manual part III (1987).

The asphalt mixtures made by use of the well graded aggregates are expected to dissipate traffic loading by both mechanisms of stone-stone contact and interlocking and mortar mechanism. As such, they are expected to have some level of capacity to transfer load at high loading temperatures. On the other hand, the gap-graded gradation 1 is expected to give rise to an asphalt mixture which dissipates traffic through mortar mechanism alone and thus expected to be sensitive to loading temperatures. The only way for these gap-graded asphalt mixtures to overcome effects of high loading temperatures is by use of bitumen with low temperature susceptibility.

5.2.3 Penetration Grade Bitumen Test Data

5.2.3.1 Introduction

Table 4.2 in chapter 4 presents the results of the penetration grade bitumen tests. The analysis of these results is as shown in Table 5.2. The results are compared by the acceptance criteria values as given in the British Standards specifications (BS 3690). Table 5.2 shows the tests used in the characterization of the penetration grade bitumen.

5.2.3.2 Penetration Test Results

The penetration test results in Table 5.2 characterize the consistencies of the acquired penetration bitumen grades. The results show that at the testing temperatures of 25^0C, the grade 180/200 penetration grade bitumen has the lowest consistency while that one of grade 60/70 has the highest consistency. A comparison of consistencies of the acquired bitumen grades obtained by testing, with the corresponding consistency range given by the manufacturer confirm the materials to be suitable for the study.

5.2.3.3 Softening Point Test Results

The softening point test results are given in Table 5.2. The results indicate the temperature at which the different bitumen grades have the same consistency in other words it is an indication of a manner in which the consistencies of different bitumen grades are affected by temperature changes. The obtained results in Table 5.2 indicate that 180/200 penetration grade bitumen has the highest temperature susceptibility while 60/70 penetration grade bitumen has the lowest temperature susceptibility. In other words, the temperature susceptibility increases from 60/70 penetration grade bitumen to 180/200 penetration grades bitumen. When the measured temperature susceptibility values are compared with their corresponding values range for each bitumen grade, (BS 3690), all bitumen grades are found acceptable for the study.

5.2.3.4 Penetration Index (PI) Results.

The penetration index (PI) is an indicator of the temperature susceptibility of a bitumen grade. Penetration index can be obtained by combining the consistency of a bitumen grade at a fixed temperature as measured by the penetration test at 25^0C, with its temperature at which a fixed consistency occurs as measured by softening point test. The penetration index results as shown in table 5.2 are obtained by use of a Nomograph. The values of penetration index obtained from the Nomograph by Pfeiffer and Van Doormaal (1936) show that the 60/70 penetration grade bitumen is less susceptible to temperature as compared to 80/100 and 180/200 penetration grade bitumens. The 180/200 penetration grade bitumen has the highest temperature susceptibility of the three grades while the 80/100 penetration grade bitumen has a medium temperature susceptibility among the three bitumen grades. These Nomograph penetration index values are within the range of -1 to +1 (BS 3690).

Table 5.2: Comparison of measured properties value with acceptance criteria values of penetration Bitumen Grades

Test	Units	Penetration grade Bitumen Properties					
		Measured values			Acceptance criteria values		
		60/70	80/100	180/200	60/70	80/100	180/200
Penetration @ 25^0C	0.1mm	65	90	190	60 to 70	80 to 100	180 to 200
Softening point (R&B)	^{0}C	50	48	42	48 to 56	45 to 52	37 to 43
Penetration index (Nomograph)	-	-0.5	-0.6	-0.9	-1 to +1	-1 to +1	-1 to +1
Specific gravity @ $25/25^0$C	-	1.04	1.06	1.06	1.01 to 1.06	1.00 to 1.05	1.00 to 1.05

5.2.3.5 Specific Gravity Test Results

The specific gravity of bitumen is the ratio of the weight of any volume of bitumen to the weight of an equal volume of water both at a specified temperature. The specific gravity values in Table

5.2 of the three penetration grade bitumen grades @ $25/25^0C$ mean both the bitumen and water were at a temperature of 25^0C. The specific gravity is normally used to make volume corrections when volume measurements are made at elevated temperatures and also as a factor in determination of voids in compacted bituminous mixtures.

5.3 Marshall Mix Design Data

Marshall Mix design procedure in this study was used for the sole purpose of obtaining total asphalt mixtures for characterization of permanent deformation of asphalt mixtures by use of triaxial compression test results. One set of the triaxial test samples was prepared using the optimum bitumen contents obtained by use of Marshall Mix design procedures.

The Marshall Mix design data is as presented in Tables 4.4(a) and 4.4(b) as well as Tables 1.1 to 1.12 and Figures 1.1.1 to 1.12.7 of Appendix 1. Table 4.4(a) shows the design mix parameters for the first approximated design optimum bitumen content while Table 4.4(b) shows the design mix parameters for the final or the actual design optimum bitumen content. The latter was used to prepare the second set of triaxial test specimens. Tables 1.1 to 1.12 in the appendix 1 indicate the Marshall Mix design data results for the four blended aggregate skeletons mixed with 60/70, 80/100 and 180/200 penetration grade bitumen. Figures 1.1.1 to 1.12.7 shows the various curves necessary to carry out and complete the Marshall Mix design procedures where the end product is the required optimum bitumen content to bind each of the four aggregate blends.

The behaviour of the corresponding Marshall Mix design curves conforms to the expected trends in the Marshall Mix design procedures (ASTM D1559). Table 4.4(b) shows the designed properties of the 12 bituminous mixtures to be investigated in this study. The optimum bitumen contents were found to be between 5.1% and 6.2% which is within the expected range for durable bituminous mixtures. A good example of such bituminous mixtures is the asphaltic concrete for surface course i.e. type 1 wearing course as per MOR&PW Roads Design Manual Part III (1987). Table 5.3 shows that well graded blended aggregates gradations 2, 3, and 4 requires higher percentages of designed optimum bitumen content than the gap-graded blended aggregate gradation 1. This is due to the fact that the well graded aggregates have got a bigger surface area as opposed to the gap-graded aggregates gradation. Among the well graded aggregate blends, blend type 4 has the highest bitumen content due to the fact that it contains highest percentage of the fine aggregates. It is observed that the optimum bitumen content increases with decrease in the consistency of bitumen for all the aggregate gradations 1, 2, 3 and 4 as shown in Table 5.3.

Voids in the mix (VIM) are the percentage of air voids in the bituminous mixture calculated as shown by equation 1.6 of appendix 1. This parameter needs to be controlled to minimize the extent of permanent deformation. High percentages of VIM are suspected to cause bleeding under heavy traffic conditions. The criteria values for VIM are 3$\pm$1% to 5$\pm$1% for surfacing materials (Jackson and Brien 1962) and 3$\pm$1% to 8$\pm$1% for road base materials (The Asphalt institute, 1994) as shown in table 4.4(b) of chapter four. Lower limits are adopted in case of light traffic while higher limits are adopted for heavy traffic scenarios. Table 4.4(b) of chapter four shows that the VIM values for the 12 number prepared samples ranges between 2.5 and 6.6 and well within the set criteria. It is therefore justifiable to characterize these prepared specimens for permanent deformation taking into consideration the fact that the asphalt mixes qualify to be those of type 1 surfacing as per Road Design Manual Part III (1987).

The aggregate voids filled with bitumen (VFB) is the percentage of voids in mixed aggregates (VMA) that is occupied by bitumen calculated as shown by equation 1.8 of appendix 1.The criteria values range of VFB is as shown in Table 4.4(b) of chapter four (75$\pm$5% to 85$\pm$5%) for asphalt concrete where the tyre pressure is 100psi or 0.7Mpa (Jackson and Brien 1962).The test results as presented in Table 4.4(b) indicates that the VFB ranges between 62% and 85% which qualifies the asphalt mix for use as type I surfacing as per Road Design Manual Part III (1987).

Voids in the mixed aggregates, (VMA) is the percentage void content in mixed aggregates calculated as shown by Equation 1.7 of appendix 1. The VMA should be controlled and the mixture produced should avoid having either the lowest or highest VMA value possible.

Table 5.3; Optimum Bitumen content for blended aggregates 1, 2, 3, and 4

Aggregates gradation	1			2			3			4		
Penetration bitumen grades	60/70	80/100	180/200	60/70	80/100	180/200	60/70	80/100	180/200	60/70	80/100	180/200
Designed optimum Bitumen content (%)	5.1	5.2	5.5	5.3	5.5	6.1	5.5	6.0	6.1	5.8	5.9	6.2

This is because the lowest values of voids content in mixed aggregates (VMA) makes a bituminous mixture not to carry sufficient amounts of bitumen. The consequences are that, the mixture will be difficult to compact and hence less durable. On the other hand, the highest values of VMA will result in mixtures which are very rich in bitumen and hence low stability. Table 4.4(b) of chapter four shows the calculated values and the recommended design criteria values. The VMA values were found to be between 15% and 19% against the recommended 16-19%. It can then be argued that the measured values were well within the recommended range as presented in Table 4.4(b) of chapter four.

Another important parameter in the prepared asphalt mixtures is the compacted density of mix (CDM) calculated as shown by equation 1.3 of appendix 1. The CDM results are important only when the spread area per tonne is taken into consideration especially during the real construction of pavements. It is found that the spread area is inversely proportional to the CDM. This means then that a mixture with high value of compacted density of mix (CDM) will result into a small spread area when compacted into a given layer thickness. In contravention, a mixture with a low value of compacted density of the mix (CDM.) will give a big spread area for a given thickness of pavement layer. The compacted density of the mix (CDM) can be used as an economy indicator of the pavement materials. The lower the value of CDM the higher the spread area when laying a layer of a given pavement thickness and hence more economical. On the other hand the higher the value of CDM, the lower the spread area when laying a layer of a given pavement thickness and hence making it to be less economical.

5.4 Triaxial Test Data

5.4.1 Introduction

In a conventional triaxial compression test, a cylindrical sample is loaded axially to failure, at a constant confining pressure. Conceptually, the peak value of the axial stress is taken as the confined compressive strength of the sample. In view of the variability of sample properties, when adequate samples are available, repeat testing is carried out to determine average values. For the purpose of this study, the peak value of the axial stress is referred to as the maximum stress. The confining pressures in this study were applied at three different levels, 0.1Mpa, 0.2Mpa and 0.4Mpa.The maximum confining stress was chosen as 0.4Mpa reason being that any confining stresses beyond this caused leakage in the triaxial cell of the test machine used.

The confining pressures are very important in providing a lateral support which prevents the sample or the pavement structure for that matter from spreading laterally. Confinement also helps to minimize the vertical displacement. In practice, confinement of the pavement structures is attained by use of kerbs and shoulders or by making sure that the layers of pavement below are wider than the layers above. Confinement is also initially attained by a way of compaction whether by use of rollers during construction or by traffic when a pavement is already in use.

Permanent deformation in paving materials develops gradually with increasing number of load applications usually appearing as longitudinal depressions in the wheel paths accompanied by small upheavals to the sides. At the first instance this is thought to occur as a result of densification and shear deformation. Densification is the decrease in volume and hence increases in density. However a study conducted by AASHO Road Test (Highway Research Board, 1962) indicated that shear deformation other than densification is the primary cause of permanent deformation. The problem of densification can be resolved by adequate compaction. It therefore makes a lot of sense to characterize the bituminous mixtures for permanent deformation by use of triaxial compression test reason being that the specimens in this test fail by shear mechanism after the maximum loading is reached. Figure 5.1 is a graphic illustration of triaxial compression test.

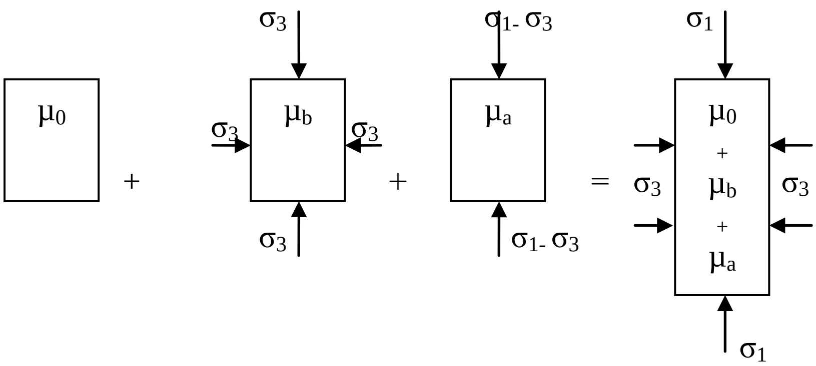


Figure 5.1; Visualization of stress state inside the triaxial cell in a typical triaxial compression test setup

Where;

σ_1 = Axial stress with its value at failure called shear stress at failure or maximum stress

σ_3 = All round pressure (confining pressure)

μ_0 = zero stress condition before the forces are applied to the asphalt specimen

μ_b = Stress state inside the triaxial cell when only the confining stress is at play

μ_a = Stress state inside the triaxial cell when the axial stress is at play.

$\mu_0 + \mu_b + \mu_a$ = The figure in the right hand of the equation shows the resultant stresses when both confining and axial stresses are at play

As mentioned earlier in chapter four, several triaxial specimens were prepared with an aim of testing them at different loading conditions so as to investigate the various factors which contribute towards permanent deformation of asphalt mixtures as outlined by the objectives in chapter one. The four parameters varied were as follows:

a) Gradation
b) Bitumen consistency
c) Bitumen content
d) Loading temperature and
e) Confining stress

Gradation is the distribution of various particle sizes within a given sample of aggregates. The desired end product (asphalt specimen) should qualify for use in pavement's top layers namely the wearing and binder course as per Kenyan flexible pavement standards. The gradation of the aggregates was done as per MOR&PW envelope with four aggregate blends 1, 2, 3, & 4 being obtained. Well graded aggregates blend's 2, 3, and 4 and a gap-graded aggregates blend 1 were obtained. The aggregate gradation is a major factor in performance of aggregates as far as the permanent deformation is concerned. Well graded aggregates transfer loading by stone-stone contact and interlocking as well as mortar mechanism which means that they still retain some ability to dissipate traffic loading even at high temperatures when the bitumen mortar becomes inconsistent, on the other hand gap graded aggregates transfer traffic loading by use of mortar mechanism only and hence expected to suffer a great loss in its ability to transfer loading at high temperatures. In this regard, the bitumen consistency will be of importance. Bitumens with low temperatures susceptibility will be a better option.

Bitumen consistency is the ability of the bitumen to maintain their binding capacity at a given temperature. Three types of penetration grade bitumen i.e. 60/70, 80/100 and 180/200, were used in this study. Penetration grade bitumen 60/70 has got ability to remain consistent at high temperatures say 50^0C and hence it is said to be less susceptible to high temperatures. Penetration grade 180/200 is highly susceptible to high temperatures and performs well only when the loading temperatures are low. Penetration grade 80/100 is of medium susceptibility. Samples made by use of these penetration grade bitumens were tested at different temperatures to assess their performance.

Bitumen content has an effect of reducing the air voids. The higher the bitumen content, the lower the air voids. Very high bitumen contents in asphalt mixtures might lead into bleeding of the mixtures under compaction or high temperatures. Very low bitumen content might reduce the workability of the asphalt mixtures hence making it difficult to compact which leads to inadequate densities or weak asphalt mixtures.

Loading temperatures are also very important in characterization of asphalt mixtures. As mentioned earlier, the consistency of bitumen is a function of loading temperatures. Loading temperatures are of great importance especially when considering the gap-graded mixtures.
From the triaxial compression tests, five major outputs are of importance in characterizing the permanent deformation of asphalt mixtures. The five parameters measured and analyzed in this study are as follows;

a) Poisson's ratio
b) Stress at initiation of plasticity
c) Stress at initiation of dilation

d) Tangent stiffness and
e) Maximum stress

5.4.2 Failure Mode of Asphalt Mixtures

From the triaxial compression test, Stress – strain curves were plotted with their shape corresponding to the one shown in Figure 5.2. All the stress – strain curves obtained from the triaxial compression test are as presented in figures 1.1.1 to 60.5.3 (900 No. curves) part of appendix 2 attached in form of CD – ROM due the large size involved. The two figures in Figure 5.2 compare the failure mode in one and two dimensional situation; originally the behaviour of the asphalt mixtures is linearly elastic represented by a straight line from the origin until point 1 in the left hand diagram. From point 1 which corresponds to ellipse 1 in the 2D case, the response is non linear but the load carrying capacity can still increase. The response is commonly known as hardening and in 1D case it looks like a curve with diminishing slope. Between parts 1 and 3 in the 2D case, this phase of response corresponds to a series of successive incremental ellipses 1 to 3.The strength (maximum as in point 3) in the 1D situation corresponds to the largest ellipse in 2D case. After this point, the strength reduces and softening is initiated. This is illustrated by descending (between point 3 and 4). In the 2D case this is represented by series of successive ellipses diminishing in size. In this study, the stress at point 1 is termed as the stress at initiation of plasticity or dilation. The gradient of the linear part of the curve from origin to point 1 is referred to as the tangent stiffness while the stress at point 3 is referred to as the maximum stress. It is from the curves in Figure 1.1.1 to Figure 60.5.3 of appendix two (see attached CD-ROM), that the five properties of the triaxial compression test as listed in section 5.4.1, were obtained. The five triaxial compression test properties for various asphalt mixtures tested at various conditions are as tabulated in Table 1.1 – 2.15 Appendix 2.

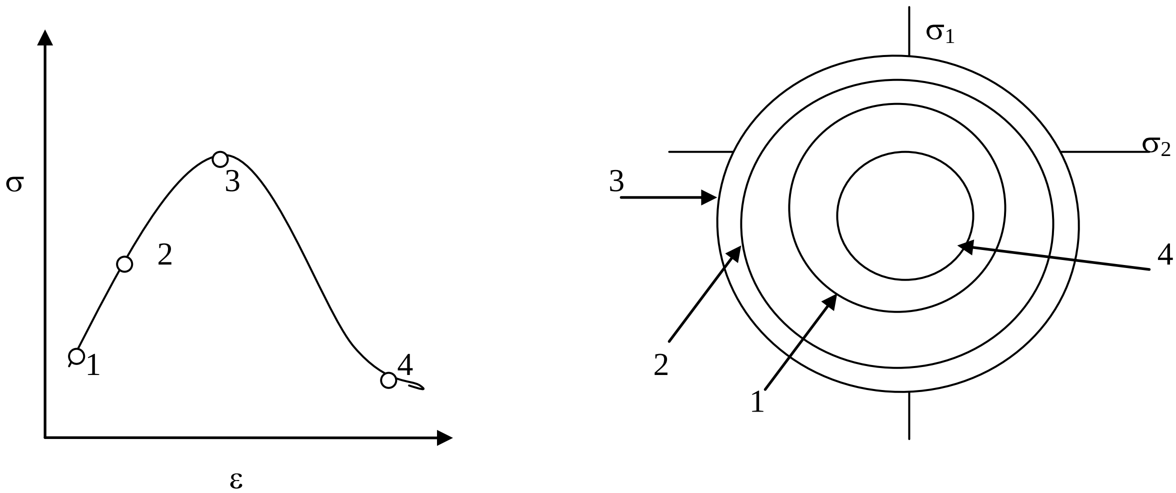


Figure 5.2; Schematization of the Desai flow surface for the 1 and 2 dimensional cases. The numbered points in the 1D diagram correspond to the ellipses in the 2D case.(Molenaar et al 2002).

5.4.3 Poisson's Ratio

Triaxial compression test was used to characterize the four asphalt mixtures prepared in this study. The triaxial compression test used in this study was the static type. From the changes in the vertical dimension (strain) measured in the test, it was possible to compute volume changes for the all the test specimens. This made it possible to work out the Poisson's ratio of the asphalt mixture specimens using equation 4 of section 3.1.3 of this research as suggested by Yoder and Witczak (1975).The equation is reproduced here as equation 5.4

$$\mu = \frac{1}{2}\left(1 - \frac{\Delta V}{\varepsilon_a V_0}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 5.4$$

Where; $\Delta V =$ Change in volume of specimen

$V_0 =$ Original volume of specimen

$\varepsilon_0 =$ Axial strain measured in the direction in which the loading is applied

The values of poisson's ratio as calculated for all the mixtures tested in the study are as shown by Tables 1.1 to 2.15 of appendix 2.The values are close together within the range of 0.503 and 0.515. Figure 5.3 show an example of the variation of the Poisson's ratio with the confinement for asphalt mixtures made using 60/70 penetration grade bitumen and tested at a loading temperature of 20^0C. From the raw data in Tables 1.1 to 2.15 of Appendix 2 and Figure 5.3, it can be deduced that the values of Poisson's ratio tend to decrease with increase in the levels of confinement. This is true for asphalt mixtures of blends 1, 3 and 4. Asphalt mixtures made of blend 2 exhibited a different trend whereby the Poisson's ratio was found to decrease as confinement increased from 0.1 to 0.2Mpa after which the values of Poisson's ratio was found to increase with increase in confinement. Low values of Poisson's ratio translates into high strains while on the other hand the high values of Poisson's ratio is an indication of low values of strains in a specimen being tested or in real pavement situation. The Poisson's ratios in this study did not demonstrate meaningful variation even after the various study conditions were varied, for example the loading temperatures, gradation, bitumen type and bitumen consistency.

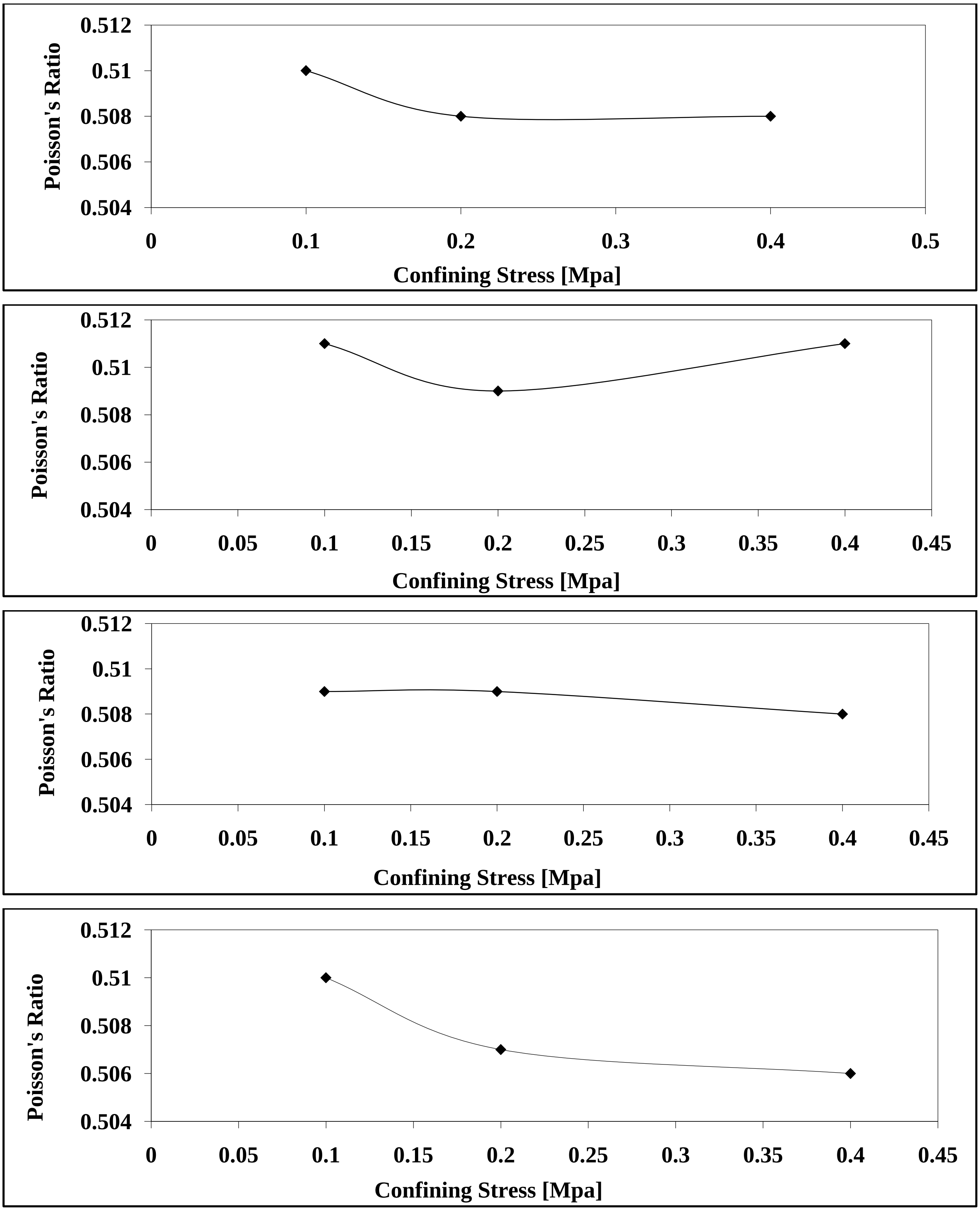


Figure 5.3; Poisson's ratio versus confining stress for asphalt mixtures 1(first plot), 2(second plot), 3(third plot) and 4(fourth plot) bound by 60/70 penetration grade bitumen and tested at 20^0C loading temperature

5.4.4 Stress at Initiation of Plasticity and Dilation

5.4.4.1 Introduction

The beginning of plasticity and dilation occurs as soon as the linear behaviour in a stress-strain curve ceases to exist. The deformation of the asphalt mixtures is recoverable after the load is removed as shown by the linear phase between origin and point 1 in a stress – strain curve, Figure 5.2. However, there reaches a point where all the deformation suffered by the asphalt specimen under test cannot be wholly recovered, at this point the stress - strain curve stops from being linear (beyond point 1 in Figure 5.2) and the material is said to start behaving plastically. The stresses at point 1 are referred to as stresses at initiation of plasticity.

In this study, the initiation of plasticity was defined by the termination of linear part of the vertical stress versus axial strain plot occurring on or before the onset of dilation. On the other hand, dilation is defined as an enlargement or expansion in bulk or extent; it is the opposite of contraction. What happens just before the initiation of dilation takes place is that the density starts to increase with increase in confinement and this occurs up to a certain point. However, change of confinement beyond a certain point has little or no influence in the volume of the test specimen. The initiation of dilation occurs when the aggregates are arranged in such a manner to occupy the minimum volume. This minimum volume decreases with increase in the confinement level leading to increase in density. Meanwhile, the minimum volume cannot be expected to decrease indefinitely with increase in confinement. At a certain confinement the aggregates will occupy the minimum possible volume and beyond this point no further increase in density can be expected. At a situation where the volume cannot decrease any longer, any confinement means extraction or bleeding of the viscous components in the asphalt mix leading to the enlargement of the specimen in a process referred to as dilation. From Figure 5.2, dilation is supposed to occur from point 2 and beyond. However, due to difficulties involved in trying to identify point 2 from the plotted stress – strain curves in this study, dilation is assumed to have started at the same time as initiation of plasticity. The initiation of plasticity and dilation marks the start of permanent deformation for the asphalt mixtures which were studied.

The initiation of plasticity and dilation marks the beginning of permanent deformation in asphalt mixtures. The researcher therefore sought to know how various factors both volumetric and environmental affects stress at initiation of plasticity and dilation. In so doing the objectives in chapter one were employed as follows;

- Effects of aggregates gradation,
- Effects of bitumen consistency,
- Effects of bitumen content,
- Effects of loading temperature and
- Effects of confining stress on stress at initiation of plasticity and dilation

5.4.4.2 Effects of Aggregate Gradation on Stress at Initiation of Plasticity and Dilation

Since the stress at the initiation of plasticity is a measure of how likely the asphalt mixtures are to undergo permanent deformation, low values of stress at initiation of plasticity are expected to be recorded for the asphalt mixtures made of gap - graded aggregates and high values when using asphalt mixtures made of the well graded aggregates. Tables 1.1 to 2.15 of Appendix 2 confirm this assertion. From the above mentioned tables and at any given temperature or bitumen consistency, the values of stress at initiation of plasticity for asphalt mix blend 1 were found to be lowest as compared to the other three asphalt mix blends 2, 3, and 4. Blend 1 aggregates are gap-graded meaning they lack the interlocking capability to dissipate loading and they can only do so using the bitumen binder. This leaves the asphalt mixtures made using the aggregate skeleton blend 1 more prone to other factors such as the loading temperature and bitumen consistency.

For the asphalt mixtures made using the well graded aggregate skeletons blend 2, 3 and 4, their stresses at initiation of plasticity varies depending on what bitumen consistency, loading temperature and bitumen content is considered. For example blend 4 asphalt mix is found to record the highest value of stress at initiation of plasticity when 60/70 penetration grade bitumen is used and the tests are conducted at 20^0C loading temperature. The same asphalt mix is found to record lower values of stress at initiation of plasticity when the same type of bitumen consistency is used but testing conducted at a higher loading temperature. The best way to find out which kind of asphalt mixture is best in resisting permanent deformation would be to check which type of mixture records the highest values of the stress at initiation of plasticity at the highest temperatures taking into consideration the bitumen consistency. Table 5.4 presents the values of stress at initiation of plasticity for the four asphalt mix blends using the three bitumen grades 60/70, 80/100 and 180/200 and tested at a loading temperature of 60^0C.

From Table 5.4, the asphalt mixtures can be ranked in terms of their ability to resist permanent deformation as asphalt mix, aggregate blend 3, 4, 2 and 1 in reducing order.

It can therefore be concluded that well graded asphalt mixtures are better in resisting permanent deformation more so at higher loading temperatures as compared to the gap-graded asphalt mixtures. This is because the well graded asphalt mixtures dissipate loading by both interlocking mechanism of the aggregates and binder effect while the gap –graded asphalt mixtures dissipate loading by use of bitumen binder. This means then that at high temperatures the bitumen binder is rendered ineffective meaning that the gap – graded aggregates are unable to dissipate traffic loads while on the other hand the well graded mixtures utilizes the aggregate interlocking mechanism to dissipate traffic loadings.

Generally the values of stress at initiation of plasticity were found to be lowest for all the asphalt mixtures made of gap – graded aggregate skeleton blend 1.This is an indication that the gap – graded asphalt mixtures are more prone to permanent deformation.

Dense, well graded aggregate gradation blends 2, 3 and 4 exhibited high resistance towards permanent deformation as compared to the open or gap – graded aggregate gradation blend 1. This is an indication that dense aggregate gradations are desirable to mitigate the effects of permanent deformation in asphalt concrete layers. Dense, well graded aggregate gradations has fewer voids and more contact points between aggregate particles than open or gap – graded mixtures. The findings are comparable to the findings of a study by Muraya (2007).

Table 5.4; Maximum values of stress at initiation of plasticity at the highest loading Temperature (60^0C)

Blend	Stress at initiation of plasticity (Mpa)			Average (Mpa)
	60/70	80/100	180/200	180/200
1	0.025	0.025	0.029	0.026
2	0.065	0.043	0.024	0.044
3	0.58	0.082	0.100	0.08
4	0.047	0.046	0.052	0.048

Note: values extracted from Tables 2.5, 2.10 and 2.15 of Appendix 2

5.4.4.3 Effects of Bitumen Grade on Stress at Initiation of Plasticity and Dilation

Bitumen consistency is a measure of stability of asphalt mixtures especially when effects of loading temperatures are considered. At high temperatures asphalt pavements deform easily though the extent of deformation is dependent on the consistency of bitumen used to prepare the asphalt mixtures as well as the gradation of the aggregate skeleton.

Three bitumen consistencies were used to prepare all the asphalt mixtures used in this study The asphalt mixtures made of aggregate blends 1 and 3 recorded the highest values of stress at initiation of plasticity when using 80/100 penetration grade bitumen followed by 180/200 and 60/70 penetration grade bitumen in reducing order as shown in Table 5.5 which is an example extracted from the main data as presented in Tables 1.1 to 2.15 of Appendix 2. From the tables, it is also observed that the asphalt mixtures made of aggregate blend 2 recorded the highest values of stress at initiation of plasticity when 60/70 penetration grade bitumen was used followed by the asphalt mixtures made of 80/100 and 180/200 penetration grade bitumen in decreasing order. For the asphalt mixtures made of aggregate blend 4, the values of stress at initiation of plasticity were highest when using 60/70 penetration grade bitumen as a binder followed by those asphalt mixtures made of 180/200 and 80/100 penetration grade bitumen in decreasing order.

When different temperatures were considered, the trend was the same as shown in Tables 5.4 and 5.5 which are extracts from the main data Tables 1.1 to 2.15 of Appendix 2. The confinement in Table 5.5 is 0.4Mpa because at this confinement maximum results were achieved.

Table 5.5; Stress at initiation of plasticity for samples tested at 20^{0}C and at a confinement of 0.4 (Mpa)

Blend	Stress at initiation of plasticity (Mpa)			Average
	60/70	80/100	180/200	
1	0.165	0.175	0.167	0.169
2	0.265	0.175	0.056	0.165
3	0.323	0.500	0.386	0.403
4	0.485	0.268	0.307	0.353
Average	0.310	0.280	0.229	

Table 5.6; Stress at initiation of plasticity for samples tested at 30^0C and at a confinement of 0.4Mpa

Blend	Stress at initiation of plasticity (Mpa)			Average
	60/70	80/100	180/200	
1	0.070	0.076	0.119	0.088
2	0.249	0.111	0.040	0.133
3	0.171	0.272	0.369	0.271
4	0.171	0.184	0.196	0.184
Average	0.165	0.161	0.151	

The stress at initiation of plasticity is a measure which can be used to evaluate the susceptibility of asphalt mixtures to undergo a permanent deformation. The higher the magnitude of stress at the initiation of plasticity, the higher the traffic loads that the asphalt mixtures are expected to carry before they can start to deform permanently. From Table 5.5 it is evident that the asphalt mixtures made of aggregate blend 3, using 80/100 penetration grade bitumen as the binder and loaded at 20^0C temperatures are likely to withstand highest loading before they start to deform plastically and hence permanently. This means that they are less prone to permanent deformation. In other words, they will be able to withstand high levels of loading before they can finally deform permanently. The magnitude of the stress at initiation of plasticity reduces with increase in loading temperature and tends to reach a limit by the time the loading temperature is 60^0C and beyond. After this temperature, the stress at initiation of plasticity remains the same meaning that the asphalt mixture draws its strength entirely from the aggregate skeleton (see Figure 5.4). Figure 5.4 shows that the asphalt mixtures made of 60/70 penetration grade bitumen records the highest values of stress at initiation of plasticity followed by the asphalt mixtures made using 80/100 and 180/200 penetration grade bitumen in order of reducing magnitude.

The above explanation means that the aggregate skeleton is a very major component in asphalt mixtures which helps a great deal in resisting permanent deformation. It is worth noting that the aggregate blend 1 which is a gap graded aggregate skeleton, records the lowest values of stress at initiation of plasticity regardless of the loading temperature or bitumen consistency. This shows that the mixtures made using the gap graded aggregate skeleton are the most prone to permanent deformation. This is because as explained earlier, the gap graded aggregates relies entirely on the binder to resist permanent deformation or to dissipate traffic loadings. It follows that the gap graded aggregate gradations do not offer any resistance to permanent deformation and the situation gets even worse at high loading temperatures and when using bitumens which are highly susceptible to high temperatures.

From the average row of Tables 5.5 and 5.6, it is evident that the highest values of stress at initiation of dilation and plasticity were recorded by asphalt mixtures made of high consistency 60/70 penetration grade bitumen. This is followed by asphalt mixtures of medium consistency 80/100 penetration grade bitumen and finally asphalt mixtures of low consistency 180/200 penetration grade bitumen. The stress at initiation of plasticity and dilation is therefore dependent on the consistency of bitumen binders. Highly consistent bitumen binders say penetration grade 60/70 is found to produce more consistent asphalt mixtures especially at high loading temperatures. The reverse is observed when using bitumens of low consistency is used.

From the average column of Table 5.5, the tested asphalt mixtures can be ranked in order of decreasing magnitude of stress at initiation of plasticity and dilation as asphalt mixture blends 3, 4, 1 and 2. It is worth noting that the mixtures in Table 5.5 were tested at 20^0C.

From the average column of Table 5.6, the tested asphalt mixtures can be ranked in order of decreasing magnitude of stress at initiation of plasticity and dilation as asphalt mixture blends 3, 4, 2 and 1. It is worth noting that the mixtures in Table 5.5 were tested at 30^0C.

From the latter two paragraphs, it is clear that the well graded asphalt mixtures records highest values of stress at initiation of dilation and plasticity while the gap – graded asphalt mixture records the lowest values of stress at initiation of plasticity and dilation especially as the loading temperature increases. Well graded asphalt mixtures transfer loading by means of aggregate interlock mechanism and the binding effect of bitumen. It means that when the loading temperatures are too high and the bitumen binders have lost their consistency the well graded asphalt mixtures continues dissipating traffic loads through aggregate interlock. Gap –graded asphalt mixtures dissipate traffic loading through the binding power of bitumen on the gap-graded asphalt mixtures. When the bitumen is rendered inconsistent by high temperatures, the gap-graded asphalt mixtures ability to dissipate traffic loading is drastically reduced leading to the permanent deformation of asphalt mixtures.

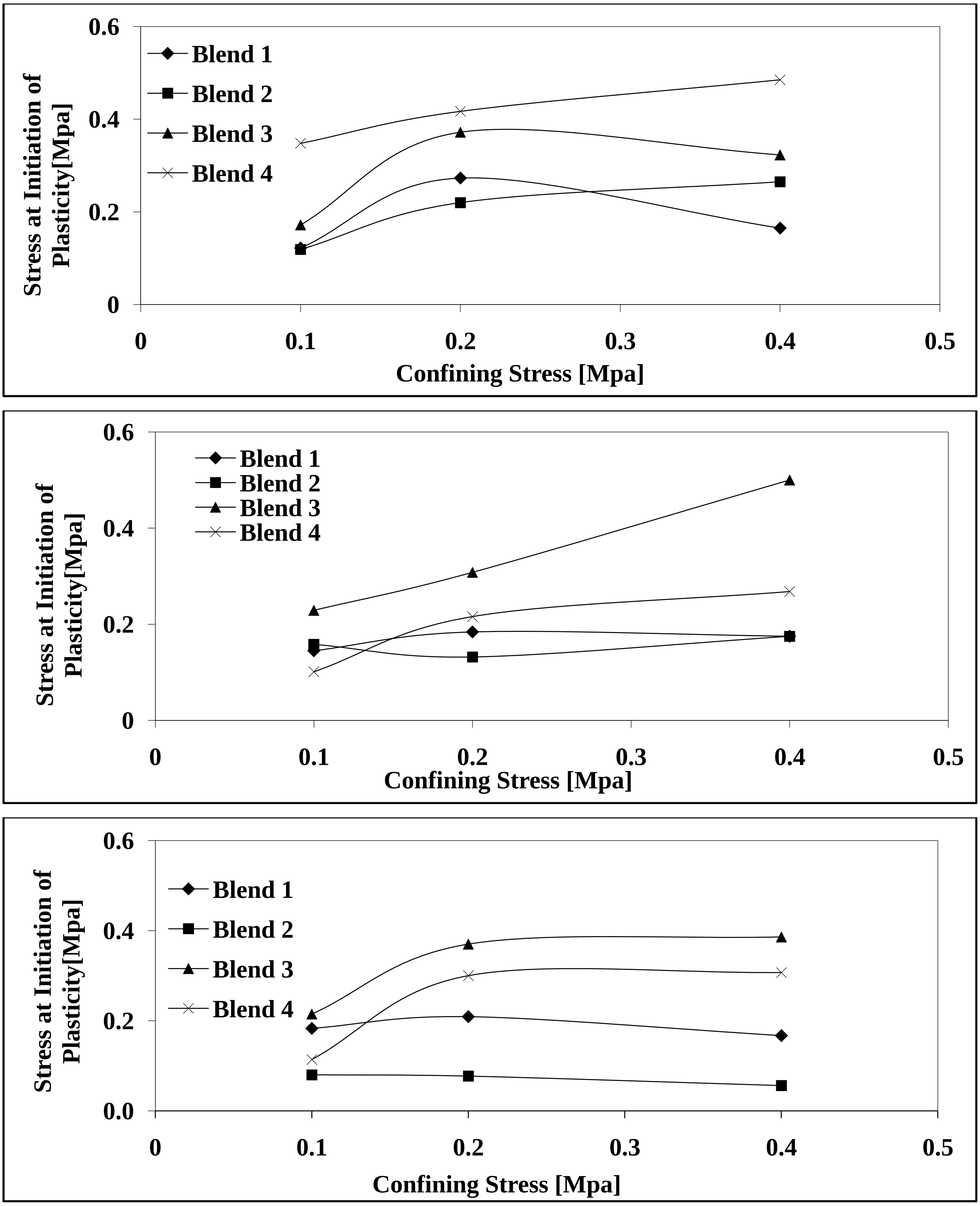


Figure 5.4; Stress at initiation of plasticity Vs the confining stress for asphalt mixture blends 1, 2, 3 and 4 mixed with optimum bitumen content penetration grade 60/70 (upper plot), 80/100 (middle plot) and 180/200 (lower plot) and tested at a loading temperature of 20^{0}C

5.4.4.4 Effects of Bitumen Content on Stress at Initiation of Plasticity and Dilation

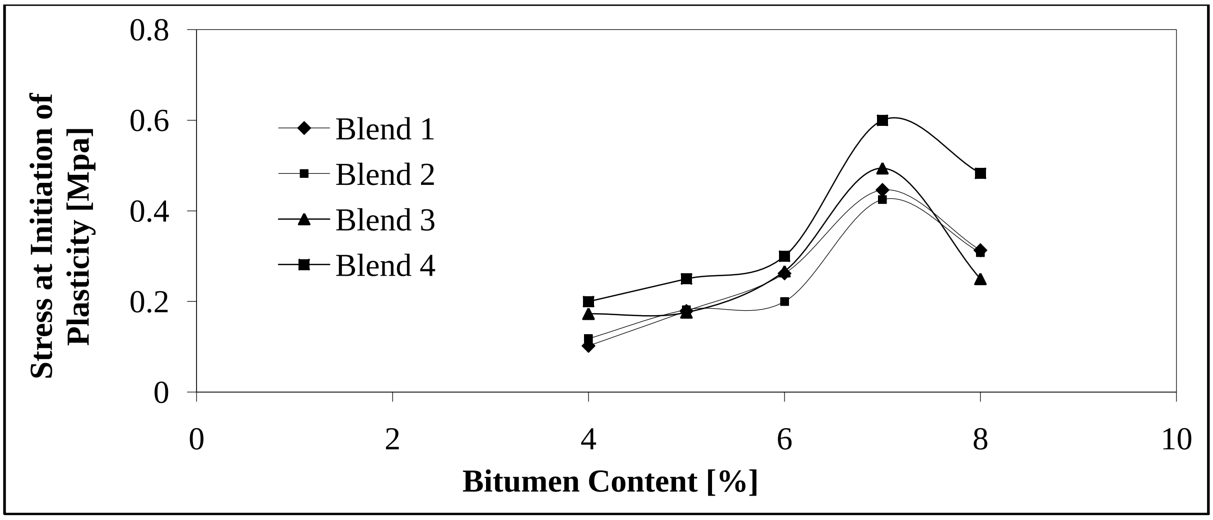
The relationship between the stress at initiation of plasticity and bitumen content is illustrated by Figure 5.5. In Figure 5.5, stress at initiation of plasticity is plotted against bitumen content for asphalt mixtures made using aggregate blends 1, 2, 3 and 4 and bound using 4, 5, 6, 7 and 8% of 60/70, 80/100 and 180/200 penetration grade bitumens, tested at a loading temperature of 20^{0}C. Specimens made of 60/70 penetration grade bitumen are represented by the upper plot, those ones made of 80/100 by the middle plot and finally samples made of 180/200 penetration grade bitumen are as shown by the lower plot of Figure 5.5.

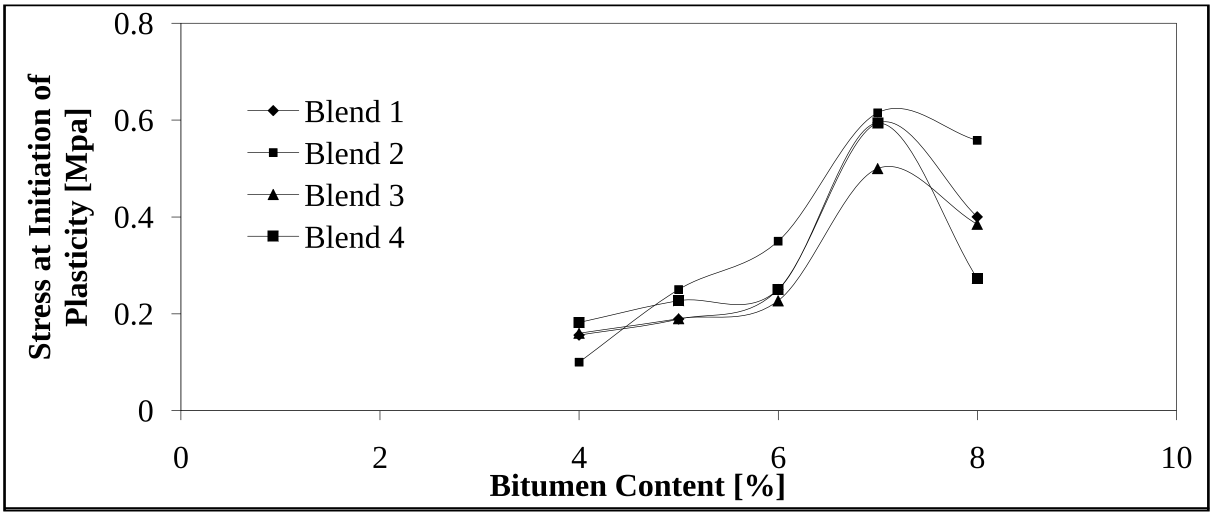
From Figure 5.5 it is observed that for all the samples made of the gap graded aggregate blend 1, the stress at initiation of plasticity increases with increase in bitumen content until optimum bitumen content is achieved beyond which the stress at initiation of plasticity decreases with increase in bitumen content. The observation can be associated with the fact that the gap-graded aggregates rely on bitumen binder to dissipate loading. However this strength can only increase up to a certain percentage of bitumen content as the air voids becomes filled with the bitumen binder until any further increase would cause the weakening of the mixture being tested. Though the values of stress at initiation of plasticity for blend 1 mixtures increases with increase in bitumen content, the highest values achieved have got the lowest magnitude as compared to the values of asphalt mixtures from the well graded aggregate blends 2, 3 and 4 owing to the fact that the aggregate skeleton of the gap-graded mixtures does not contribute towards resistance of permanent deformation.

Stress at initiation of plasticity for the asphalt mixtures made of the well graded aggregate skeletons (blends 2, 3 and 4) increased with increase in bitumen content until the optimum bitumen content was reached beyond which the stress at initiation of plasticity started to decrease. The rate at which the stress at initiation of plasticity increased with increase in bitumen content for the well graded asphalt mixtures varied depending on degree of interlock between the various aggregate gradations. Asphalt blend 2 was found to be least susceptible to permanent deformation followed by blend 4 and 3 respectively.

From Figure 5.6 and considering the upper curve for asphalt specimens made of 60/70 penetration grade bitumen, it was found that asphalt mixture blends 2 and 3 recorded increase of stress at initiation of plasticity with increase in bitumen content up to a certain limit (optimum bitumen content), beyond which the stress at initiation of plasticity started to decrease with increase in bitumen content. The same observation was made from the asphalt mixtures made of aggregate blends 3 and 4 at 80/100 and 180/200 penetration grade bitumens. The values of stress at initiation of plasticity in the above mentioned cases for the well graded aggregate blends were higher than those recorded for the gap-graded aggregate blend 1. This is an indication that the asphalt mixtures made of the well graded aggregate skeletons are capable of withstanding high stresses at elastic stage of deformation before they can start to undergo the plastic deformation and eventually the permanent deformation.

From the latter discussion, it can be concluded that permanent deformation of asphalt mixtures is dependent on the bitumen content. Too much or too low bitumen content will lead to increased vulnerability of asphalt mixtures to permanent deformation. Too much bitumen leads into a situation where most of the air voids are filled with bitumen and the aggregates literally float in the bitumen binder leading to bleeding in pavements and hence permanent deformation. Too little bitumen content will result into a mixture with very low workability meaning it becomes impossible to compact adequately. Without adequate compaction, the strength of the asphalt mixtures is greatly reduced making them to be more vulnerable to permanent deformation. The correct amount of bitumen should therefore be optimum to avoid the scenario in the latter sentences.





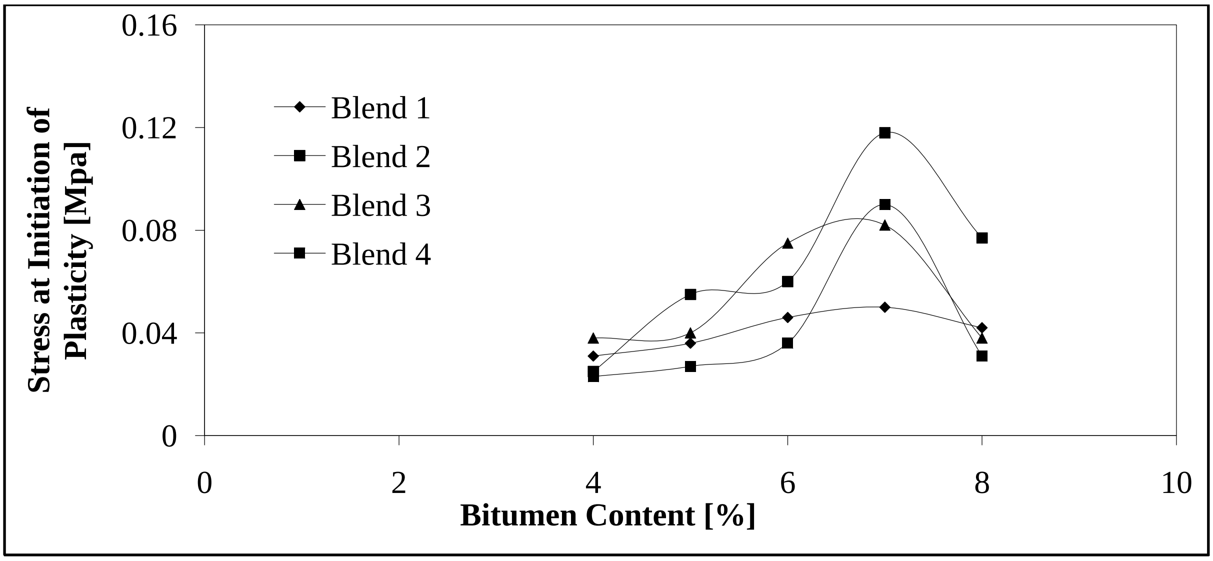


Figure 5.5; Stress at initiation of plasticity Vs bitumen content for asphalt mixtures 1, 2, 3 and 4 mixed with 60/70 (upper plot), 80/100 (middle plot) and 180/200 (lower plot) penetration grade bitumens and tested at a loading temperature of 20^0C

5.4.4.5 Effects of Loading Temperature on Stress at Initiation of Plasticity and Dilation

The values for the stress at initiation of plasticity were found to decrease with increase in temperature for all the four asphalt mixes investigated in this study. The stress at initiation of plasticity is in other words the maximum elastic stress. It is the maximum stress that a given asphalt mix specimen can bear before they start to deform permanently. It is a well known fact that at high loading temperatures asphalt mixtures tend to deform easily and this is best explained by the reduction of the recorded maximum stress as the loading temperatures increases. The rate of decrease is however subject to many more factors such as the bitumen content, bitumen consistency and aggregate gradation. The variation of the stress at initiation of plasticity with the loading temperature is as shown in Tables 1.1 to 2.15 of Appendix 2.

Figure 5.6 is an example of variation of stress at initiation of plasticity considering a specimen made of aggregate blend 2, mixed with 80/100 penetration grade bitumen and tested at 0.4Mpa confining stress. From the plots, it is clear that the stress at initiation of plasticity tends to reach a limit as the loading temperatures increase towards 60^0C and beyond. This is explained by the fact that at loading temperature of 60^0C and above, the effect of the bitumen content and type in the strength of the total asphalt mix dwindles as the bitumen binder becomes increasingly inconsistent. At such a point, the strength of the asphalt mix is solely contributed by the aggregate skeleton and hence any change in temperature would not have an effect on the point at which the asphalt mixture losses its elasticity. From figure 5.6 it is observed that the highest magnitude of stress at initiation of plasticity was recorded for asphalt mixtures with 60/70 penetration grade bitumen binder. This was followed by asphalt mixtures with 80/100 and 180/200 penetration grade bitumens in order of reducing magnitude.

From the above discussion, it can therefore be concluded that the permanent deformation of asphalt mixtures is dependent on the loading temperatures. High loading temperature leads to high rate of permanent deformation. The way to overcome the problem associated with high loading temperature will be to encourage use of the well graded asphalt mixtures bound by bitumens of high consistency.

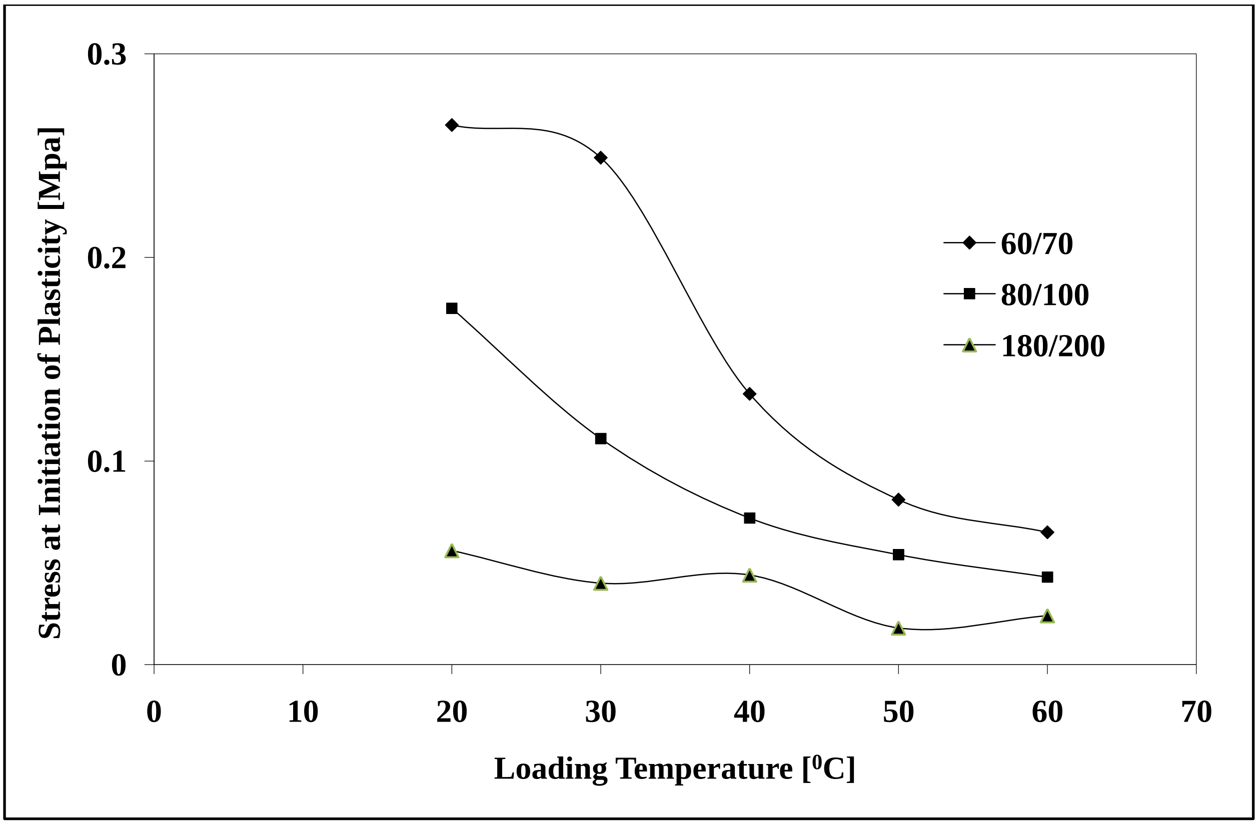


Figure 5.6; Stress at initiation of plasticity versus the loading temperature for blend 2 aggregates bound by 60/70, 80/100 and 180/200 penetration grade bitumen and tested at 0.4Mpa confining stress.

5.4.4.6 Effects of Confining Stress on Stress at Initiation of Plasticity and Dilation

Figure 5.4 is a plot of stress at initiation of plasticity versus confining stress. It is an example of asphalt mixture blends 1, 2, 3 and 4 bound using 60/70, 80/100, 180/200 penetration grade bitumens and tested at a loading temperature of 20^{0}C. From the three plots it is observed that the stress at initiation of plasticity generally increases with increase in confining stress. This is an indication that well compacted mixtures have got an improved ability to resist permanent deformation or at least delay the stresses at which the plasticity is initiated which means that more axle loads are allowed before the permanent deformation starts to take place.

When using 60/70 penetration grade bitumen, blend 4 records the highest stress at initiation of plasticity followed by blends 3, 2 and 1 in reducing order, as the confining stress is increased from 0.1mpa to 0.4Mpa. For mixtures made using 80/100 penetration grade bitumen, asphalt mixtures made of aggregate blend 3 recorded the highest values of stress at initiation of plasticity followed by those ones of aggregate blends 4, 2, and 1 in order of reducing magnitude as the confining pressure was increased from 0.1 to 0.4Mpa. When using 180/200 penetration grade bitumen, blend 3 was found to record the highest values of stress at initiation of plasticity followed by blends 4, 1 and 2 in order of reducing magnitude as the confinement was increased from 0.1 to 0.4Mpa. (See Figure 5.4 and Tables 1.1 to 2.15 of appendix 2). The trend was found to be similar for all the other tests conducted at 30^0C, 40^0C, 50^0C and 60^0C loading temperatures. From Figure 5.4, it is observed that asphalt mixtures made using 80/100 penetration grade bitumen recorded higher values of stress at initiation of plasticity than those recorded for 60/70 and 180/200 penetration grade bitumens, as the confining stresses were increased from 0.1 to 0.4Mpa.

Asphalt mixtures made using 60/70 penetration grade bitumen recorded the second highest values while, 180/200 penetration grade mixes recorded the lowest values. The shape of the curves in Figure 5.4 suggests that a limiting value of stress at initiation of plasticity is achieved as confinement approaches a maximum of 0.4Mpa.This is due to loss of density influence as the maximum density is achieved beyond which the hardening process is initiated.

From the preceding paragraphs it can be concluded that the permanent deformation is dependent on the confining stress. The higher the confining stress, the lower the permanent deformation of asphalt mixtures. Confinement helps in overcoming the lateral stresses hence reducing the amount of permanent deformation. In real pavements, confinement is provided for by allowing for kerbs or adequate shoulders.

5.4.5 Tangent Stiffness

5.4.5.1 Introduction

Tangent stiffness refers to the slope of the linear part of a stress versus strain curve. It is a measure of the elastic modulus for the asphalt specimens tested using triaxial compression test. The curves of stress-strain relationship were found to be initially linear before the plasticity could be initiated. The reason behind this is as explained earlier where it was highlighted that the asphalt mixtures posses ability to initially recover from the deformations caused by traffic loading. However, there reaches a point where the rate of deformation is higher than the rate of recovery and at that point the plastic phase is initiated. The trend continues until the asphalt material is unable to recover any lost strength and at such point, the material is said to undergo permanent deformation until the total failure occurs.

Tangent stiffness is used in this study as one of the parameters in characterizing asphalt mixtures for permanent deformation. Tangent stiffness is an elastic property and therefore its value may give an indication of how soon the permanent deformation of asphalt mixtures may start after the loading is applied. The higher the values of tangent stiffness, the better the asphalt mix is in resisting permanent deformation.

The study investigated the relationship between the tangent stiffness and the loading temperature, bitumen content, aggregate gradation, bitumen consistency and the confining tress as aligned in the objectives of this study in chapter one. This was achieved by preparing the asphalt specimens with varying volumetric contents and testing them at varying environmental conditions.

The results of triaxial test were therefore analyzed under the following sub –headings as far as the tangent stiffness is concerned;

- Effects of aggregates gradation ,
- Effects of bitumen consistency ,
- Effects of bitumen content ,
- Effects of loading temperature and
- Effects of confining stress on tangent stiffness

5.4.5.2 Effects of Aggregate Gradation on Tangent Stiffness

In order to be able to compare the tangent stiffness in relation to the aggregate skeleton we must hold the bitumen consistency, confinement, bitumen content and loading temperatures as constants. When the test results of the asphalt mix specimens prepared using 60/70 penetration grade bitumen are observed, it is found that asphalt mix blend 4 records the highest values of the tangent stiffness followed by the asphalt mix blends 3,2 and 1 in order of the reducing magnitude. For asphalt mix specimens made using 80/100 penetration grade bitumen, blend 3 recorded the highest values of the tangent stiffness followed by asphalt mix blends 4, 2 and 1 in order of the reducing magnitude. For asphalt mix specimens that were prepared using 180/200, penetration grade bitumen, it was found that asphalt mix blend 3 recorded the highest values of the tangent stiffness followed asphalt mix blends 4, 1 and 2 in order of reducing magnitude,(see the results as presented in Tables 1.1 to 2.15 of Appendix 2).

From the observations in the latter paragraph, it is clear that the well graded asphalt mixtures record high values of tangent stiffness than their gap –graded counterparts at similar loading conditions i.e. loading temperatures and confining stresses. This is an indication that well graded asphalt mixtures are better placed in resisting permanent deformation than the gap –graded asphalt mixtures. The level of resistance of permanent deformation among the well graded asphalt mixtures also varies depending on the level of interlocking by the aggregate skeleton. The higher the interlocking capability, the higher the resistance to permanent deformation an asphalt mixture is.

From the preceding discussion, it can be concluded that the permanent deformation of asphalt mixtures is dependent on the gradation of aggregate skeletons.

5.4.5.3 Effects of Bitumen Consistency on Tangent Stiffness

For asphalt mixtures made using aggregate blends 1 and 3 and tested at the same temperature, the highest tangent stiffness was recorded for specimens prepared using 80/100 penetration grade bitumen followed at the second position by the mixtures made using 180/200 penetration grade bitumen. The asphalt mix specimens which were prepared using 60/70 penetration grade bitumen recorded the lowest values of the tangent stiffness, (see Tables 1.1 to 2.15 Appendix 2).

For specimens made using aggregate blend 2 the highest values of tangent stiffness were recorded when using 60/70 penetration grade bitumen as the binder, followed by 80/100 and 180/200 penetration grade bitumens in order of reducing magnitude, (see Tables 1.1 to 2.15 appendix 2).

For aggregate blend 4, the asphalt mixtures that were prepared using 60/70 penetration grade bitumen recorded the highest values of tangent stiffness followed by those prepared using 180/200 and 80/100 penetration grade bitumen in the order of reducing magnitude, (see the results presented in Table 1.1 to 2.15 of Appendix 2).

It is observed that the tangent stiffness of similar asphalt mixtures reduces with increase in the loading temperature. The magnitude of tangent stiffness reduces with increase in inconsistency of bitumens. i.e. The magnitude of tangent stiffness for asphalt mixtures made using highly consistent 60/70 penetration grade bitumen are higher than those ones of mixtures made using highly inconsistent 180/200 penetration grade bitumen. This is so especially at high loading temperatures of 60^0C.

Even though the tangent stiffness increased with increase in bitumen consistency, the magnitude of increase varied depending on the aggregates gradation with the well graded asphalt types 2, 3 and 4 recording higher values than their gap-graded counterpart blend 1.

The tangent stiffness also reduced with increase in loading temperature. The reduction was more for asphalt made of 180/200 penetration grade bitumen than for asphalts made of 60/70 penetration grade bitumen. In other words asphalt mixtures made of inconsistent bitumens (180/200) were more prone to permanent deformation than the asphalt mixtures made of consistent bitumens (60/70).

5.4.5.4 Effects of Bitumen Content on Tangent Stiffness

The tangent stiffness of the asphalt mixtures was found to reduce with increase in bitumen content up to a certain point thought to be the optimum bitumen content after which it starts to increase again with the increase in bitumen content. Figure 5.7 is a plot of tangent stiffness versus bitumen content for asphalt mix made of aggregate blend 1 and bound by 60/70 penetration grade bitumen and tested at 0.1Mpa confinement. The full results are captured by Tables 1.1 to 2.15 Appendix 2.

For the total asphalt mixtures which were prepared using the optimum bitumen content, it was found that the mixtures which had the highest values of the optimum bitumen content, recorded the highest values of the tangent stiffness see Tables 2.1 to 2.15 Appendix 2. This is an indication that high bitumen contents in asphalt mixtures will lead to permanent deformation occurring earlier than expected i.e. after a few axle loads and the condition might be worse at high loading temperatures on asphalt pavements made of inconsistent bitumen binder. That is why it is important to obtain the correct optimum bitumen content, the one which allows for adequate voids in the mix. Too low bitumen content will lead into too much voids in the mix which leads to reduced workability making it hard for adequate compaction and thus weak pavements prone to permanent deformation. Too much bitumen on the other hand will lead to reduced amount of voids causing the aggregates in the asphalt mix to float in the bitumen which will result into the bleeding of asphalt pavements especially during high temperatures.

From the discussion in the latter paragraphs, it can be concluded that permanent deformation is dependent on the bitumen content of asphalt pavements. The higher the bitumen content, the more prone the asphalt mixtures are to permanent deformation. Also too low bitumen content would cause the asphalt mixtures to be prone to permanent deformation .To overcome this problem the bitumen content should be designed to be optimum. In case of this study the optimum bitumen content was found to be about 6%.

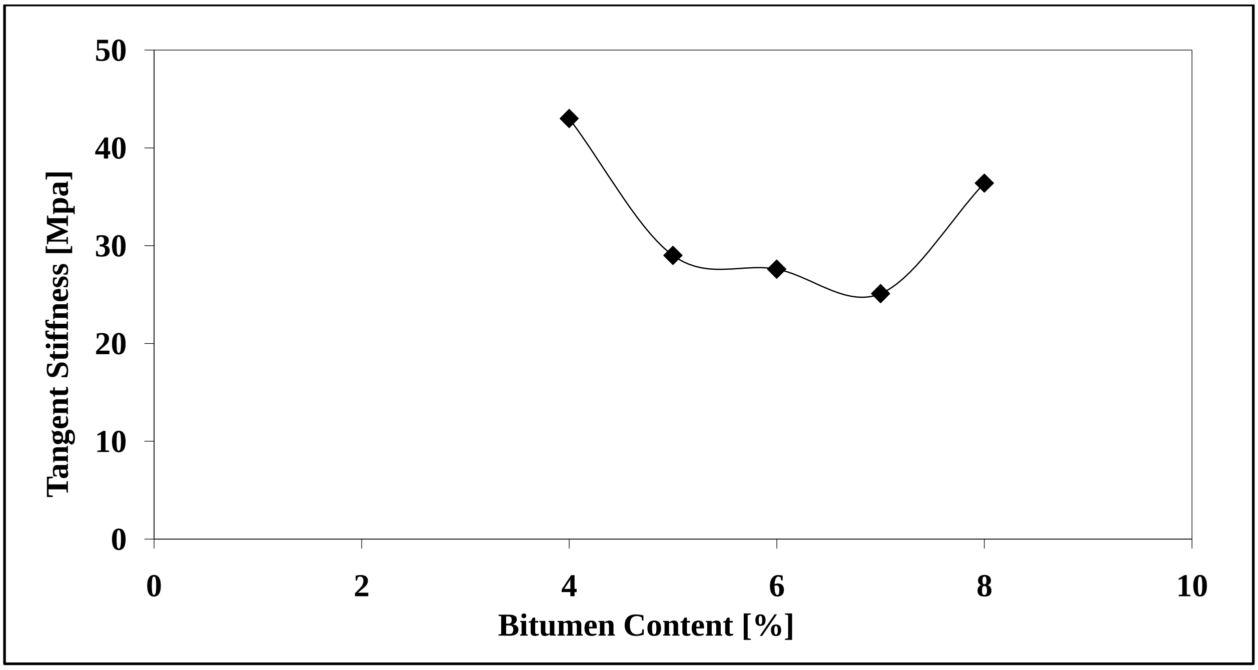


Figure 5.7; Tangent Stiffness versus bitumen content for asphalt mix made of aggregate blend 1 bound by 60/70 penetration grade bitumen and tested at 0.1Mpa confinement

5.4.5.5 Effects of Loading Temperature on Tangent Stiffness

From Tables 1.1 to 2.15 of Appendix 2, the tabulated results indicate that the tangent stiffness generally reduces with increase in the loading temperatures regardless of all the other test conditions that were varied in the study. Figure 5.8 is an example of tangent stiffness variation with the loading temperatures for asphalt mix made of aggregate blend 1 at optimum bitumen content for the three penetration grade bitumens 60/70, 80/100 and 180/200.

From Figure 5.8, it is clear that the values of the tangent stiffness seem to converge by the time the loading temperature approaches 60^0C. This is as a result of loss of the bitumen consistency at high temperatures. When the bitumen binder loses consistency at high temperatures, the aggregate skeleton interlocking remains the only mode by which the asphalt mixtures can dissipate the loading. This means that the well graded asphalt mixtures are better positioned to resist permanent deformation due to their ability to dissipate loading by use of aggregate interlocking mechanism which means they can go on dissipating the traffic loading long after the bitumen binder is rendered useless by high loading temperatures. The opposite is true for the gap-graded asphalt mixtures which can dissipate loading only by use of the strength offered by the bitumen binder.

From the preceding paragraphs, it can be concluded that the permanent deformation of asphalt mixtures is dependent on the loading temperature of the asphalt pavements. This would mean that pavements will be generally weak and prone to permanent deformation during the hot seasons of the year. The problem can be overcome by use of well graded asphalt mixtures where the aggregate skeleton will continue resisting permanent deformation after the bitumen binder is rendered inconsistent by the effects of high temperatures.

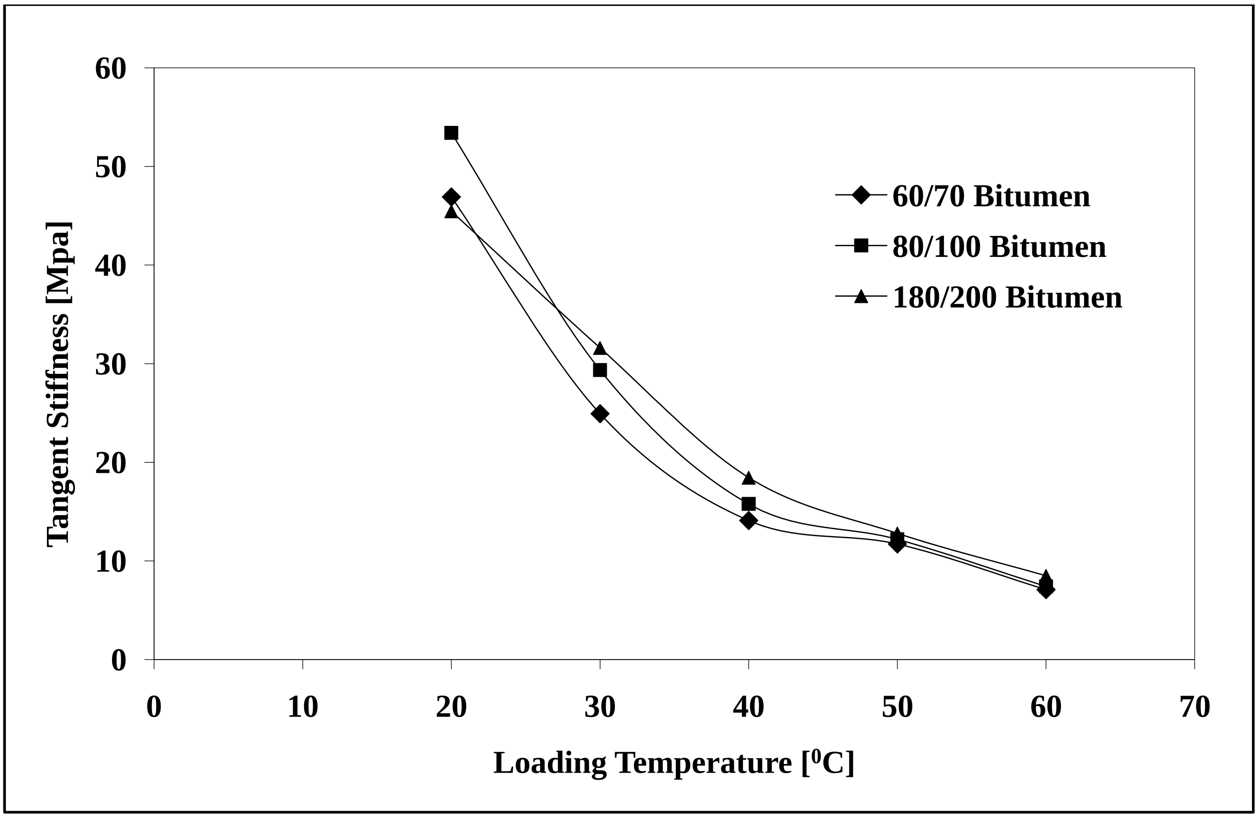


Figure 5.8; Tangent Stiffness versus loading temperature for asphalt mix made of aggregate blend 1 at optimum bitumen content, penetration grades 60/70, 80/100 and 180/200 and tested at 0.4Mpa confinement

5.4.5.6 Effects of Confining Stress on Tangent Stiffness

Generally, for all the asphalt mixtures prepared and tested using the triaxial compression test, the values of the tangent stiffness were found to increase with increase in the confining stress regardless of all the other test conditions being investigated. Figure 5.9 is an illustration of how the tangent stiffness for all the four asphalt mixtures prepared using 60/70 penetration grade bitumen and tested at 20^0C varies with variation of the confining stress. From Figure 5.9, it can be seen that blend 1 generally records the lowest values of the tangent stiffness at any given confinement while blend 4 records the highest values of the tangent stiffness at any given confinement.

From Figure 5.9 it is clear that the tangent stiffness increases with increase in confining stresses. This means that confinement will help in reducing the vulnerability of asphalt pavement from permanent deformation. Confinement helps in restraining the pavement laterally which helps in eliminating or reducing the lateral deformation of asphalt pavements. In real pavements, confinement may be catered for by kerbs and shoulders.

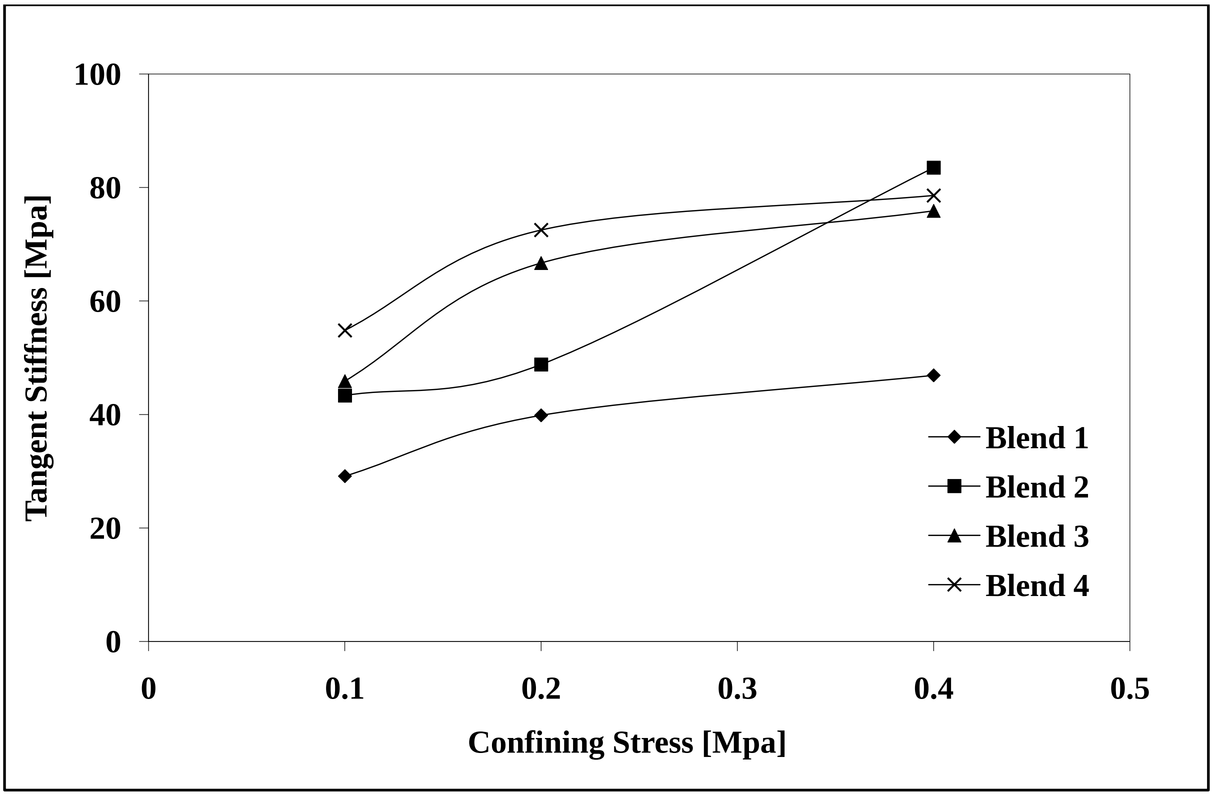


Figure 5.9; Tangent Stiffness versus Confining stress for asphalt mix made of aggregate blends 1, 2, 3 and 4, bound by 60/70 penetration grade bitumen and tested at 20^0C loading temperature

5.4.6 Maximum Stress

5.4.6.1 Introduction

Maximum stress refers to the stress at which the triaxial specimen under triaxial compression test fails. It is a measure or an indication of how much stress an asphalt mixture can withstand before they can completely deform permanently. In practice, a complete permanent deformation is physically displayed in form of wheel tracks or cracks. Wheel tracks will occur when the bitumen used is highly susceptible to high temperatures and at the same time the loading temperatures are high. Permanent deformation will occur inform cracks if the bitumen used is less susceptible to high temperatures and at the same time the loading temperatures are too low which makes the asphalt mix to be brittle hence prone to cracking. The maximum stress can therefore be collectively used to mean the compressive strength of an asphalt mix. It is a measure of resistance of the permanent deformation by asphalt mixtures. The higher the magnitude of the maximum stress, the higher the ability of an asphalt mixture to resist permanent deformation.

The maximum stress test data in this study was analyzed with an aim of achieving the objectives set in chapter one. The analysis sought to find out the effects of volumetric and environmental factors on the maximum stress as a measure of permanent deformation. The analysis was conducted under the following sub-tittles;

- Effects of aggregates gradation,
- Effects of bitumen consistency ,
- Effects of bitumen content ,
- Effects of loading temperature and
- Effects of confining stress on maximum stress

5.4.6.2 Effects of aggregates gradation on maximum stress

For all the asphalt specimens made using 60/70 penetration grade bitumen, asphalt mix blend 1 recorded the lowest values of maximum stress at any given temperature. (See table 5. 7 in conjunction with Tables 1.1 to 1.5 and 2.1 to 2.5 of Appendix 2).

The asphalt mix blend 1 is made of gap graded aggregate skeleton blend 1 as mentioned earlier in this chapter. As such, it relies wholly on the bitumen binder to resist permanent deformation from the test loading or traffic loading in real pavements. The aggregate skeleton lacks the interlocking ability of various aggregate particle sizes within it to robustly resist loading hence low maximum stresses are required to cause failure. The gap –graded asphalt mix blend1 also recorded the lowest values of maximum stresses at the highest loading temperature of 60^0C among all the asphalt mix specimens made using 60/70 penetration grade bitumen. As a result of loss of bitumen consistency with increase in loading temperature, it means that all the asphalt mixtures which derive their strength from the bitumen binder suffer a reduction in strength at high temperatures. The loss in strength emanates in form of low values of maximum stress which is the case for the gap-graded asphalt mixtures. This explains why the lowest values of maximum stress were recorded at the highest temperatures of 60^0C for gap – graded asphalt mix blend 1.

For all the asphalt mix specimens made using the well graded aggregate skeletons 2, 3, and 4 high values of maximum stress were recorded. Among these well graded asphalt mixtures, asphalt mix blend 4 recorded the highest values of maximum stresses especially at low temperatures. This can be attributed to two facts; one being that among the three well graded asphalt mixtures, asphalt blend 4 has the highest optimum bitumen content. The second factor is that it is well graded meaning both the binder and the aggregate skeleton contributes to the strength of the asphalt mix specimen. This type of mix is strong but appears to be brittle at low temperatures and might fail in form of cracking at low temperatures when used for construction of roads in areas where the loading temperatures are low.

Among the three well graded asphalt mix blends 2, 3 and 4, asphalt blend 2 recorded the highest values of maximum stress at the highest loading temperature of 60^0C. This is because it had the lowest bitumen content among the three well graded asphalt mixtures, which means that when the bitumen losses its consistency the aggregate skeleton will take the full charge of dissipating the loads. The lower bitumen content means that the strength lost due to bitumen inconsistency was of less extent just as the bitumen content was low. This type of mix will be appropriate for highly trafficked roads where the loading temperatures are high.

The maximum stress results of asphalt mix specimens prepared using 80/100 penetration grade bitumen are as presented in Table 5.8. It is observed that the specimens made of the gap graded asphalt mix blend 1 recorded the highest values of maximum stress at low temperatures (between 20^0C and 30^0C) than their well graded asphalt mix blends 2, 3 and 4.

However, at high temperatures gap graded asphalt mix blend 1 recorded the lowest values of maximum stress as compared to well graded asphalt blends 2, 3 and 4. This is due to the fact that the 80/100 penetration grade bitumen has a medium consistency and it is expected to perform well at lower temperatures of say 20^0C and 30^0C; however at high temperatures of say 60^0C the bitumen consistency is low meaning reduction of the binding strength leading to loss of strength in gap graded mix blend 1. At high temperatures, the well graded asphalt mixtures recorded higher values of maximum stress than the gap graded asphalt mixtures since after the loss of binding strength due to increased bitumen inconsistency, the well graded aggregates used its ability to interlock to continue resisting the loading effects. The gap graded asphalt mix blend 1 could also have exhibited high values of maximum stresses at low temperatures due to the fact that it had the lowest optimum bitumen content as compared to the well graded asphalt mix blends 2, 3, and 4.

The maximum stress results for asphalt mixtures specimens prepared using 180/200 penetration grade bitumen are as presented in Table 5.9. It is observed that the maximum stress values are higher at low temperatures for the gap graded asphalt mix blend 1 than for the well graded asphalt mix blends 2, 3 and 4. The reason attributed to this observation is that the bitumen contents for the gap graded asphalt specimens is lower than those ones of the well graded asphalt mixtures. Secondly, the bitumen binders are stable at low temperatures (20^0C to 30^0C) and offers a substantial amount of strength to the gap graded asphalt mixtures. The maximum stress values of the gap-graded asphalt mix blend 1 are lowest at the highest loading temperature as compared to the maximum stress values for the well graded asphalt mix blends 2, 3 and 4. This is because at high temperatures, the bitumens are highly inconsistent denying the gap graded mix the strength acquired from the binder .On the other hand the well graded asphalt mixtures continues to record high values of maximum stress due to their interlocking capability which means they continue to resist a certain fraction of permanent deformation hence the reasonably high values of maximum stress continued to be recorded.

From the preceding observations and discussions, it is clear that the permanent deformation of asphalt mixtures in form of the maximum stress is dependent on the aggregate gradation. Well graded asphalt mixtures appear to perform well especially at high temperatures and highly inconsistent bitumens. On the other hand, gap-graded asphalt mixtures performs dismally especially at high temperatures and highly inconsistent bitumens.

5.4.6.3 Effects of bitumen consistency on maximum stress

By observing the results presented by Tables 5.7, 5.8 and 5.9 and Figure 5.10, it is found that all the four blended asphalt mix specimens recorded highest maximum stresses when the binder was made of 80/100 penetration grade bitumen. The 180/200 penetration grade bitumen asphalt specimens recorded the second highest values of the maximum stress while the 60/70 penetration grade bitumen specimens recorded the lowest values of the maximum stress. The optimum bitumen contents for all the four blended asphalt mix specimens increase with the lowest values recorded for 60/70 penetration grade bitumen specimens while the highest values were recorded for 180/200 penetration grade bitumen specimens. It would appear that the strongest asphalt specimens were obtained by use of medium consistency bitumen i.e. 80/100 penetration grade bitumen.

For the gap-graded asphalt mix blend 1, the highest values of maximum stress at 60^0C loading temperature were recorded for specimens prepared using 80/100 penetration grade bitumen while the lowest values of the maximum stress at 60^0C loading temperature was recorded for the specimens made using the 60/70 penetration grade bitumen. For the well graded asphalt mix blends 2, 3 and 4 the maximum stress at the highest loading temperatures (60^0C), were recorded for specimens made using 180/200 penetration grade bitumen .The values of the maximum stress were found to decrease in order of asphalt specimens prepared using 60/70, 80/100 and 180/200 penetration grade bitumen, respectively.

From the discussion of the latter paragraphs, it can be concluded that the permanent deformation of asphalt mixtures is independent on the consistency of asphalt mixtures. It is found that the higher the consistency of the bitumens, the lower the permanent deformation of asphalt mixtures especially at high loading temperatures.

5.4.6.4 Effects of bitumen content on maximum stress

The values of maximum stresses for all the blended asphalt mixtures appeared to increase with increase in bitumen content up to a certain maximum bitumen content (assumed to be the optimum bitumen content) beyond which the values started to reduce. See Tables 1.1 to 1.15 Appendix 2.

The trend holds even with variation of all the other test conditions such as the loading temperature, gradation bitumen consistency and confinement. Air voids and voids in the aggregate skeleton filled with bitumen play a significant role in the resistance of permanent deformation. If the voids filled with bitumen are exceeded beyond a certain limit over filling occurs as a result of extension of volume of bitumen at high temperatures. When the overfilling occurs, the strength of the asphalt mixtures is significantly reduced. In real pavements, the problem shows itself in form of bleeding especially at high loading temperatures. The resulting effect is that the permanent deformation is accelerated especially when the asphalt mix is made of gap-graded aggregate skeleton .In this study the same was observed when the bitumen content surpasses the optimum values designed by use of Marshall Procedure. At high bitumen contents say 7% and 8%, the values of maximum stress starts to reduce meaning a weaker asphalt mix with reduced ability to resist permanent deformation. The reduction of maximum stress beyond the optimum bitumen content was found to increase significantly with increase in loading temperatures. Too low bitumen content will reduce the workability of the asphalt mixtures leading to asphalt mixtures with reduced densities and hence strength.

From the above observations and discussions, it is evident that permanent deformation of asphalt mixtures is dependent on the bitumen consistency. Optimum values of bitumen content should be used in preparation of asphalt mixtures capable of resisting permanent deformation.

5. 4.6.5 Effects of loading temperature on maximum stress

From Tables 5.7, 5.8 and 5.9, it is observed that the value of the maximum stress for all the four blends of aggregates reduces with increase in loading temperature. This is true regardless of the level of confinement, aggregate skeleton, bitumen type and consistency among others. This is because as the loading temperatures increases, the bitumen binder looses their consistency and hence their binding abilities leaving the aggregate skeleton to resist permanent deformation from the applied loads on their own. Without the much needed bitumen binder, the raw aggregates have got less capability or no capability at all to resist permanent deformation.

On average, from Tables 5.7, 5.8 and 5.9, it can be deduced that by increasing the loading temperatures from 20^0C to 60^0C at an optimum bitumen content and constant confinement of 0.4Mpa, the maximum stresses of asphalt mix blends 1, 2, 3 and 4 reduce by 86, 77, 80 and 84% respectively. This indicates that the four asphalt mixtures in this study can be ranked in order of decreasing ability to resist permanent deformations as 2, 3, 4 and 1.

Figure 5.10 is a plot of maximum stress versus the loading temperature. The curves compare the variation of the maximum stress with the loading temperatures for asphalt mix blends 1, 2, 3 and 4 specimens prepared using 60/70, 80/100 and 180/200 penetration grade bitumens and at a constant confinement of 0.4Mpa.From the curves, it is observed that the maximum stress reduces with increase in loading temperatures regardless of the bitumen consistency and aggregate gradation. The values of the maximum stress tend to converge to a constant value at a temperature of 60^0C and beyond.60/70 penetration grade bitumen asphalt mixtures records the lowest value of maximum stress at 20^0C while 80/100 penetration grade asphalt mixtures records the highest value of maximum stress at the same loading temperature.

It can be concluded that the permanent deformation of asphalt mixtures is dependent on the loading temperature. Asphalt mixtures are found to be more prone to permanent deformation at high loading temperatures, and when made of inconsistent bitumen and gap –graded aggregate skeletons.

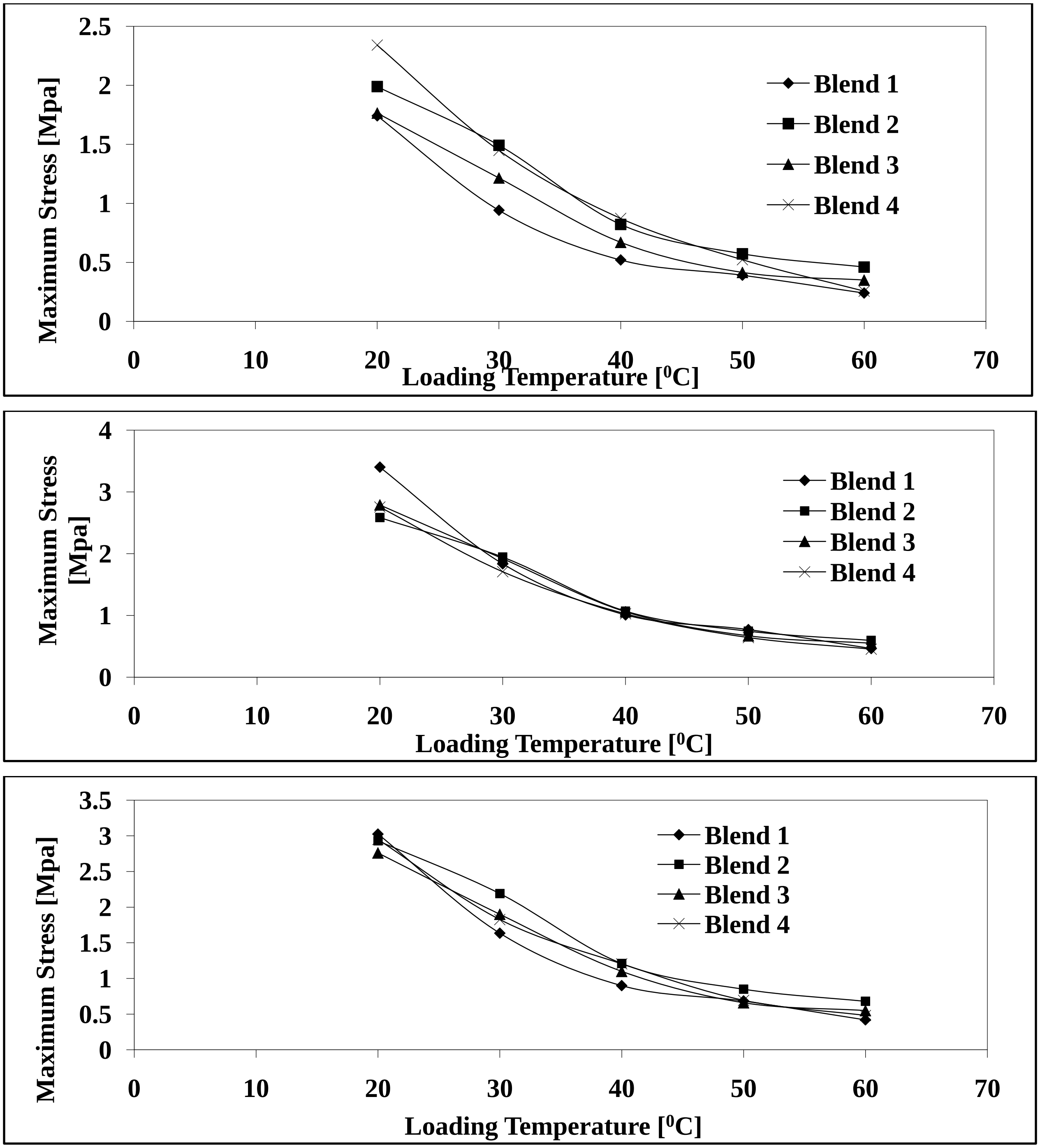


Figure 5.10; Vertical maximum stress versus loading temperature for asphalt mixtures 1, 2, 3 and 4 bound by 60/70 (upper plot), 80/100 (middle plot) and 180/200 (lower plot) penetration grade bitumens and tested at 0.4Mpa confining pressure

5.4.6.6 Effects of confining stress on maximum stress

The maximum stress was found to generally increase with increase in the confinement regardless of all the other factors which were varied in the study; say temperature, gradation and bitumen type and content. This is presented in Tables 1.1 to 1.5 and 2.1 to 2.15 of Appendix 2.

The asphalt mixtures behave in this manner mainly because of two reasons. Reason number one being that the confinement increases the compaction of the specimens leading to more dense specimens with higher compressive strength. The second reason is that confinement aids in preventing the lateral spreading of the vertical asphalt specimens by offering a horizontal support all round the specimen making sure that the only effective force in action is the vertical one which in turn leads to all the displacements being vertical. This way, the asphalt mixture specimens are able to withstand more compressive forces.

Though the general trend is as explained in the latter paragraph, the magnitude of the maximum stress varies depending on the gradation of the aggregates, the bitumen content and type and the loading temperature. The results are as presented in Tables 5.7, 5.8 and 5.9 for the total asphalt mixtures. (The three tables are extracts from Table 2.1 to Table 2.15 of appendix 2).

From the preceding paragraphs, it can be concluded that the permanent deformation of asphalt mixtures is dependent on the confining stress. The higher the confining stress, the lower the permanent deformation. On top of designing strong asphalt mixtures, proper kerbs and adequate shoulders should be provided for in order make sure that adequate confinement is provided for so as to minimize the permanent deformation of asphalt mixtures.

Table 5.7; Comparison of maximum stresses for a total mixture made using 60/70 penetration grade bitumen and tested at different temperatures

Aggregates blends	Loading Temperature (^{0}C)	Optimum Bitumen content (%)	Maximum stress at different confinement levels (Mpa)		
			0.1	0.2	0.4
1	20	5.1	0.628	1.215	1.741
	30	5.1	0.344	0.659	0.941
	40	5.1	0.192	0.360	0.520
	50	5.1	0.142	0.278	0.391
	60	5.1	0.097	0.165	0.240
2	20	5.3	0.900	1.189	1.989
	30	5.3	0.790	0.896	1.492
	40	5.3	0.432	0.495	0.821
	50	5.3	0.307	0.347	0.572
	60	5.3	0.243	0.276	0.460
3	20	5.5	1.345	1.505	1.765
	30	5.5	0.930	1.040	1.215
	40	5.5	0.510	0.570	0.670
	50	5.5	0.320	0.360	0.415
	60	5.5	0.270	0.300	0.350
4	20	5.8	1.230	1.592	2.34
	30	5.8	0.766	0.988	1.448
	40	5.8	0.462	0.594	0.872
	50	5.8	0.278	0.360	0.522
	60	5.8	0.134	0.178	0.256

Table 5.8; Comparison of maximum stresses for total mixtures made using 80/100 penetration grade bitumen and tested at different temperatures.

Aggregates blends	Loading Temperature (^{0}C)	Optimum Bitumen content	Maximum stress at different confinement levels (Mpa)		
			0.1	0.2	0.4
1	20	5.2	2.274	2.832	3.402
	30	5.2	1.228	1.532	1.836
	40	5.2	0.672	0.844	1.008
	50	5.2	0.510	0.640	0.770
	60	5.2	0.316	0.388	0.464
2	20	5.5	1.860	1.940	2.585
	30	5.5	1.395	1.450	1.945
	40	5.5	0.770	0.800	1.070
	50	5.5	0.540	0.560	0.745
	60	5.5	0.400	0.450	0.595
3	20	6.0	1.630	2.110	2.790
	30	6.0	1.130	1.460	1.920
	40	6.0	0.620	0.800	1.060
	50	6.0	0.390	0.500	0.670
	60	6.0	0.320	0.420	0.550
4	20	5.9	1.614	2.168	2.755
	30	5.9	0.993	1.345	1.708
	40	5.9	0.608	0.812	1.027
	50	5.9	0.373	0.512	0.642
	60	5.9	0.268	0.361	0.454

Table 5.9; Comparison of maximum stresses for total mixtures made using 180/200 penetration grade bitumen and tested at different loading temperatures

Aggregates blends	Loading Temperature (^{0}C)	Optimum Bitumen content	Maximum stress at different confinement levels (Mpa)		
			0.1	0.2	0.4
1	20	5.5	1.920	2.450	3.025
	30	5.5	1.035	1.325	1.635
	40	5.5	0.575	0.725	0.900
	50	5.5	0.435	0.550	0.685
	60	5.5	0.265	0.340	0.420
2	20	6.1	2.080	2.170	2.930
	30	6.1	1.560	1.630	2.190
	40	6.1	0.860	0.890	1.210
	50	6.1	0.600	0.630	0.850
	60	6.1	0.480	0.500	0.680
3	20	6.1	1.620	2.090	2.760
	30	6.1	1.120	1.440	1.900
	40	6.1	0.640	0.830	1.100
	50	6.1	0.390	0.500	0.660
	60	6.1	0.320	0.410	0.550
4	20	6.2	1.776	2.286	2.948
	30	6.2	1.106	1.420	1.826
	40	6.2	0.658	0.854	1.205
	50	6.2	0.416	0.534	0.694
	60	6.2	0.290	0.374	0.482

5.5 Modeling the Permanent Deformation of Asphalt Mixtures

5.5.1 Introduction

As outlined in chapter one of this research, the sixth objective was to model the permanent deformation of asphalt mixtures. Modeling of permanent deformation in this study was done by use of constitutive equations or laws. Constitutive laws are correlations between pairs of measured quantities or characteristics of asphalt mixtures. The quantities or the measured characteristics were determined using the triaxial compression test conducted on identical asphalt specimens of 38mm diameter and 76mm height.

The characteristics determined by the triaxial compression test included the tangent stiffness and the maximum stress. In both cases the characteristics were determined at three levels of confining stress namely 0.1Mpa, 0.2Mpa and 0.4Mpa as well as five temperature levels namely 20, 30, 40, 50 and 60^{0}C.

The tangent stiffness was correlated with the loading temperatures using a natural logarithmic relationship as discussed in section 5.5.2 while the maximum stress was correlated using power law as discussed in section 5.5.3.

5.5.2 Modeling the Tangent Stiffness in Respect to the Loading Temperature

A constitutive law of the form shown in Equation 5.5 was used to model the tangent stiffness in respect to the loading temperature. This is because constitutive models are simple whereby only two strength parameters are used to describe plastic behaviour of asphalt mixtures. Researchers have indicated by means of triaxial compression tests that stress combinations causing failure in asphalt mixtures agree well with the constitutive models (Kok, 2009).

$$\ln T_S = C_1 + k_1 \ln T \qquad \text{……5.5}$$

Where; T_S = tangent stiffness (Mpa)

C_1, k_1 = Materials constants

T = loading temperature (^{0}C)

Equation 5.5 is a natural logarithmic relationship between the tangent stiffness and the loading temperature and was developed using the least square regression analysis. The results are as presented in Table 5.10 with a total number of thirty six equations being obtained.

The high values of coefficient of determination (R^2) as shown in Table 5.10 is an indication that the thirty six equations obtained give a highly reliable prediction of tangent stiffness for all the four asphalt mixtures 1, 2, 3 and 4, at any given temperature between 20^0C and 60^0C using the three bitumen grades of 60/70, 80/100 and 180/200 with the triaxial compression test conducted at either 0.1, 0.2 or 0.4Mpa confinement. This means that a Road Engineer is able to estimate the expected values of the tangent stiffness for a given asphalt mix at a given temperature and levels of confinement giving him or her an advantage of at least knowing how likely an asphalt mixture is to start deforming permanently and take the necessary design adjustment in order to come up with a strong mix able to resist the permanent deformation.

Table 5.10 ; Constitutive equations for modelling Tangent Stiffness (T_S) in respect to loading Temperature (T)

Asphalt Mix Blend	Bitumen Grade	Confining Stress [Mpa]	Coefficient of Determination (R^2)	Constitutive Equation
1	60/70	0.1	0.975	$lnT_S = 8.503 - 1.696ln\mathrm{T}$
		0.2	0.963	$lnT_S = 8.613 - 1.643ln\mathrm{T}$
		0.4	0.972	$lnT_S = 8.854 - 1.666ln\mathrm{T}$
	80/100	0.1	0.978	$lnT_S = 8.764 - 1.732ln\mathrm{T}$
		0.2	0.973	$lnT_S = 9.369 - 1.826ln\mathrm{T}$
		0.4	0.967	$lnT_S = 9.317 - 1.767ln\mathrm{T}$
	180/200	0.1	0.981	$lnT_S = 8.811 - 1.756ln\mathrm{T}$
		0.2	0.971	$lnT_S = 9.197 - 1.787ln\mathrm{T}$
		0.4	0.972	$lnT_S = 9.060 - 1.672ln\mathrm{T}$
2	60/70	0.1	0.975	$lnT_S = 8.078 - 1.419ln\mathrm{T}$
		0.2	0.974	$lnT_S = 8.297 - 1.436ln\mathrm{T}$
		0.4	0.977	$lnT_S = 8.787 - 1.423ln\mathrm{T}$
	80/100	0.1	0.965	$lnT_S = 6.958 - 1.197ln\mathrm{T}$
		0.2	0.966	$lnT_S = 7.679 - 1.377ln\mathrm{T}$
		0.4	0.983	$lnT_S = 8.117 - 1.426ln\mathrm{T}$
	180/200	0.1	0.976	$lnT_S = 7.453 - 1.430ln\mathrm{T}$
		0.2	0.978	$lnT_S = 7.212 - 1.435ln\mathrm{T}$
		0.4	0.972	$lnT_S = 6.515 - 1.165ln\mathrm{T}$

Table 5.10 Continued;

Asphalt Mix Blend	Bitumen Grade	Confining Stress [Mpa]	Coefficient of Determination (R^2)	Constitutive Equation
3	60/70	0.1	0.982	$lnT_S = 8.313 - 1.476ln\mathrm{T}$
		0.2	0.983	$lnT_S = 9.308 - 1.697ln\mathrm{T}$
		0.4	0.974	$lnT_S = 9.123 - 1.565ln\mathrm{T}$
	80/100	0.1	0.965	$lnT_S = 8.970 - 1.559ln\mathrm{T}$
		0.2	0.968	$lnT_S = 9.343 - 1.581ln\mathrm{T}$
		0.4	0.975	$lnT_S = 9.445 - 1.571ln\mathrm{T}$
	180/200	0.1	0.977	$lnT_S = 9.694 - 1.782l\mathrm{nT}$
		0.2	0.965	$lnT_S = 9.373 - 1.588ln\mathrm{T}$
		0.4	0.984	$lnT_S = 9.445 - 1.573ln\mathrm{T}$
4	60/70	0.1	0.985	$lnT_S = 10.283 - 1.946ln\mathrm{T}$
		0.2	0.963	$lnT_S = 10.044 - 1.965ln\mathrm{T}$
		0.4	0.972	$lnT_S = 9.671 - 1.776ln\mathrm{T}$
	80/100	0.1	0.975	$lnT_S = 8.355 - 1.618ln\mathrm{T}$
		0.2	0.963	$lnT_S = 9.026 - 1.664ln\mathrm{T}$
		0.4	0.972	$lnT_S = 9.619 - 1.666ln\mathrm{T}$
	180/200	0.1	0.975	$lnT_S = 8.599 - 1.647ln\mathrm{T}$
		0.2	0.963	$lnT_S = 9.057 - 1.654ln\mathrm{T}$
		0.4	0.972	$lnT_S = 9.499 - 1.634ln\mathrm{T}$

5.5.3 Modelling the Maximum Stress in Respect to the Confining Stress

A power model is used to describe the maximum stress results obtained from the triaxial compression test for the various asphalt mixtures which were prepared. The power model shown in Equation 5.6 is used to describe the relationship between the maximum stress and the confining stress. The other way that the results of the maximum stress can be modeled is by use of the Mohr - Coulomb failure criterion (see Equation 5.7). However, the power model gave more realistic data fit in comparison to Mohr - Coulomb failure criterion reason being the fact that all the material fractions that could result in cohesion were omitted during the preparation of aggregate skeleton specimens and as a result, all the cohesive strength was provided by bitumen binder.

$$f_{ca} = a\sigma_3^b \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \ 5.6$$

Where;

f_{ca} = Maximum stress.

σ_3 = Confining stress.

a, b = Model parameters.

$$\tau = C + \sigma\tan\theta \ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \ 5.7$$

Where;

τ = Shear strength or maximum stress at failure.

σ = Normal stress on failure plane.

C = Soil cohesive resistance.

$\mathrm{Tan}\theta$ = Angle of shearing stress resistance of soil.

The power model Equation 5.6 can be re-written in natural logarithm format as in equation 5.8

$$\ln f_{ca} = \ln a + b\ln\sigma_3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \ 5.8$$

The components of Equation 5.7 can then be matched with those of equation 5.8 as follows;

$\tau = \ln f_{ca}$,

$C = \ln a$,

$\tan\theta = b$,

$\sigma = \ln\sigma_3$.

Using the results of the 12 total mixtures (as presented in Tables 2.1 to 2.15 of Appendix 2) that were prepared using the Marshall Procedures (ASTM D1559) and tested using triaxial compression test, sixty equations were obtained as presented in Table 5.11. From this table, it is observed that the power model parameters *a* and *b* reduce with increase in the loading temperature. The power model parameters *a* and *b* were highest for asphalt mix specimens made using 80/100 penetration grade bitumen than for the asphalt specimens prepared using 60/70 and 180/200 penetration grade bitumen. In other words, the values of *a* and *b* were highest for asphalt specimens made using 80/100 penetration grade bitumen followed by those made using 180/100 and 60/70 penetration grade bitumen respectively. This is because bitumen grade 80/100 is of medium consistency and hence it has moderate temperature susceptibility which makes it more appropriate for temperature range used in this study. The sixty power model equations obtained are able to reliably predict maximum stress at a given confinement and temperature for the asphalt mixtures prepared using aggregate blends 1, 2, 3 and 4 bound by 60/70, 80/100 or 180/200 penetration grade bitumens owing to the high values of coefficient of determination (R^2) as well as the result of plots in Figure 5.11.The models agree with the findings of other studies like the one conducted by Muraya, (2007).From the 60 models, $f_{ca} = 1.275\sigma_3^{0.237}$ is the best model since it had the highest coefficient of determination as compared to the other models.

Figure 5.11 shows an example of a data fit of the maximum stress based on a power model and the actual measured values for asphalt specimen blends 1, 2, 3 and 4 and tested at a loading temperature of 20^0C. Figure 5.11 shows that the vertical stress required to fail the asphalt mix blend 4 is higher than the one required to for asphalt blends 1, 2 and 3 and that the four asphalt mixes can be ranked in order of decreasing ultimate vertical stress as blend 4, blend 2, blend 1 followed by blend 3. The plot made by use of the modeled results closely matches the plot made by use of measured results.

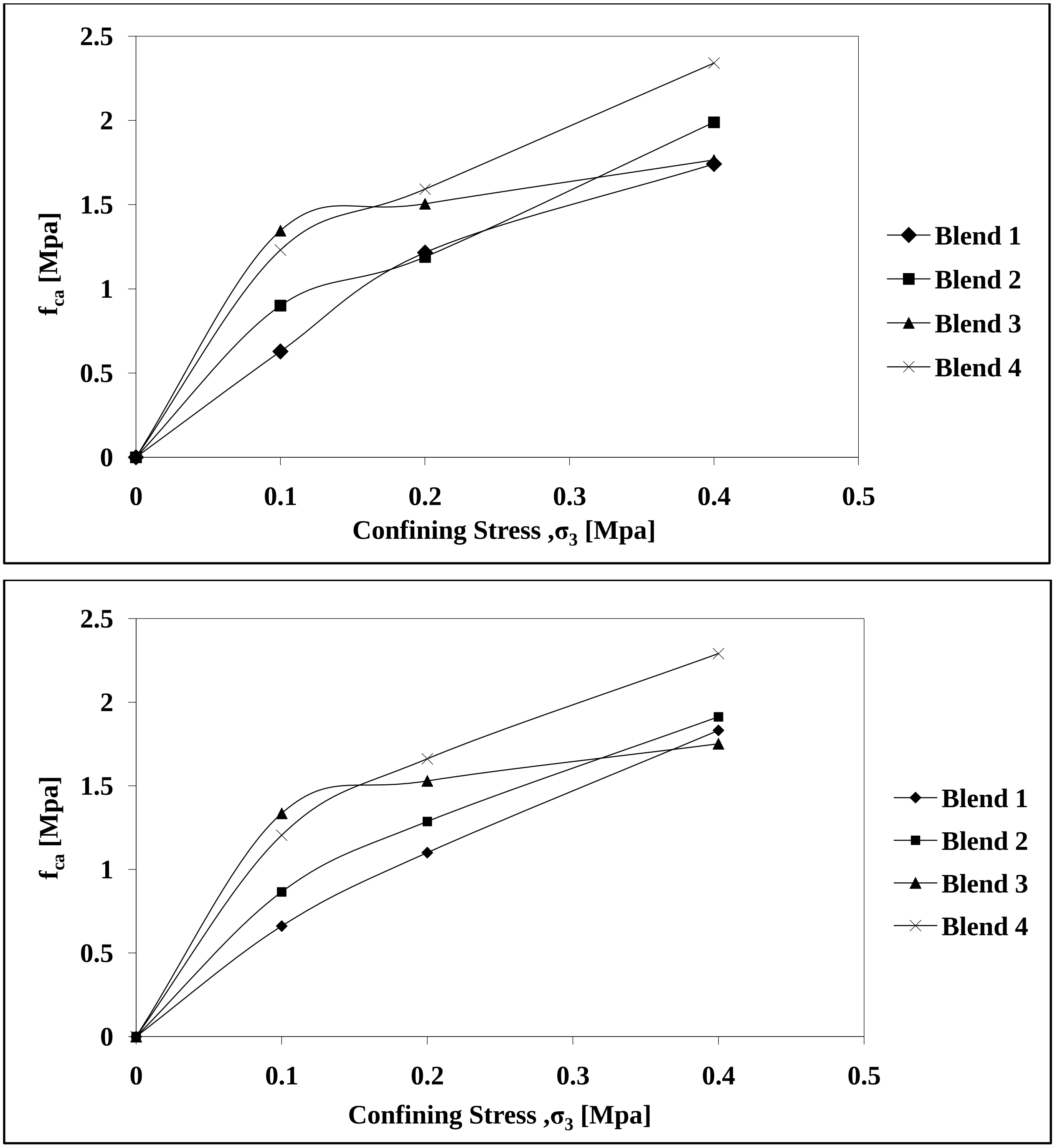


Figure 5.11; Measured (upper graph) and modeled (lower graph) maximum vertical stress f_{ca}, for aggregate blends 1, 2, 3 and 4 bound by 60/70 penetration grade bitumen and tested at a loading temperature of 20^0C

Table 5.11 ; Power model equations for asphalt mix blends 1, 2,3 and 4 at various loading temperatures

Asphalt mix blend	Loading temp (^{0}C)	Bitumen grade						Power model equation for the three bitumen grades [$f_{ca} = a\sigma_3^b$]		
		60/70		80/100		180/200				
		Model parameters		Model parameters		Model parameters		60/70	80/100	180/200
		a	b	a	b	a	b			
1	20	3.591	0.735	4.467	0.291	4.108	0.328	$f_{ca} = 3.591\sigma_3^{0.735}$	$f_{ca} = 4.467\sigma_3^{0.291}$	$f_{ca} = 4.108\sigma_3^{0.328}$
	30	1.922	0.726	2.411	0.290	2.226	0.330	$f_{ca} = 1.922\sigma_3^{0.726}$	$f_{ca} = 2.411\sigma_3^{0.290}$	$f_{ca} = 2.226\sigma_3^{0.330}$
	40	1.049	0.719	1.329	0.292	1.213	0.323	$f_{ca} = 1.049\sigma_3^{0.719}$	$f_{ca} = 1.329\sigma_3^{0.292}$	$f_{ca} = 1.213\sigma_3^{0.323}$
	50	0.807	0.731	1.018	0.297	0.927	0.328	$f_{ca} = 0.807\sigma_3^{0.731}$	$f_{ca} = 1.018\sigma_3^{0.297}$	$f_{ca} = 0.927\sigma_3^{0.328}$
	60	0.448	0.653	0.601	0.277	0.573	0.332	$f_{ca} = 0.448\sigma_3^{0.653}$	$f_{ca} = 0.601\sigma_3^{0.277}$	$f_{ca} = 0.573\sigma_3^{0.332}$
2	20	3.23	0.572	3.085	0.237	3.520	0.247	$f_{ca} = 3.23\sigma_3^{0.572}$	$f_{ca} = 3.085\sigma_3^{0.237}$	$f_{ca} = 3.520\sigma_3^{0.247}$
	30	2.131	0.459	2.322	0.240	2.628	0.245	$f_{ca} = 2.131\sigma_3^{0.459}$	$f_{ca} = 2.322\sigma_3^{0.240}$	$f_{ca} = 2.628\sigma_3^{0.245}$
	40	1.180	0.463	1.275	0.237	1.449	0.246	$f_{ca} = 1.18\sigma_3^{0.463}$	$f_{ca} = 1.275\sigma_3^{0.237}$	$f_{ca} = 1.449\sigma_3^{0.246}$
	50	0.810	0.449	0.884	0.232	1.026	0.251	$f_{ca} = 0.81\sigma_3^{0.449}$	$f_{ca} = 0.884\sigma_3^{0.232}$	$f_{ca} = 1.026\sigma_3^{0.251}$
	60	0.658	0.460	0.753	0.286	0.819	0.251	$f_{ca} = 0.658\sigma_3^{0..460}$	$f_{ca} = 0.753\sigma_3^{0.286}$	$f_{ca} = 0.819\sigma_3^{0.251}$
3	20	2.096	0.196	3.966	0.388	3.910	0.384	$f_{ca} = 2.096\sigma_3^{0.196}$	$f_{ca} = 3.966\sigma_3^{0.388}$	$f_{ca} = 3.910\sigma_3^{0.384}$
	30	1.439	0.193	2.718	0.382	2.683	0.381	$f_{ca} = 1.439\sigma_3^{0.193}$	$f_{ca} = 2.718\sigma_3^{0.382}$	$f_{ca} = 2.683\sigma_3^{0.381}$
	40	0.796	0.197	1.504	0.387	1.568	0.391	$f_{ca} = 0.796\sigma_3^{0.197}$	$f_{ca} = 1.504\sigma_3^{0.387}$	$f_{ca} = 1.568\sigma_3^{0.391}$

	Loading temp (^{0}C)	Bitumen grade						Power model equation for the three bitumen grades [$f_{ca} = a\sigma_3^b$]		
		60/70		80/100		180/200				
		Model parameters		Model parameters		Model parameters		60/70	80/100	180/200
		a	b	a	b	a	b			
	50	0.491	0.188	0.951	0.390	0.930	0.379	$f_{ca} = 0.491\sigma_3^{0.188}$	$f_{ca} = 0.951\sigma_3^{0.390}$	$f_{ca} = 0.930\sigma_3^{0.379}$
	60	0.412	0.187	0.787	0.391	0.781	0.391	$f_{ca} = 0.412\sigma_3^{0.187}$	$f_{ca} = 0.787\sigma_3^{0.391}$	$f_{ca} = 0.781\sigma_3^{0.391}$
4	20	3.504	0.464	3.960	0.386	4.120	0.366	$f_{ca} = 3.504\sigma_3^{0.464}$	$f_{ca} = 3.96\sigma_3^{0.386}$	$f_{ca} = 4.120\sigma_3^{0.366}$
	30	2.159	0.459	2.471	0.391	2.543	0.362	$f_{ca} = 2.159\sigma_3^{0.459}$	$f_{ca} = 2.471\sigma_3^{0.391}$	$f_{ca} = 2.543\sigma_3^{0.362}$
	40	1.297	0.458	1.467	0.378	1.773	0.436	$f_{ca} = 1.297\sigma_3^{0.458}$	$f_{ca} = 1.467\sigma_3^{0.378}$	$f_{ca} = 1.773\sigma_3^{0.436}$
	50	0.777	0.454	0.933	0.392	0.971	0.369	$f_{ca} = 0.777\sigma_3^{0.454}$	$f_{ca} = 0.933\sigma_3^{0.392}$	$f_{ca} = 0.971\sigma_3^{0.369}$
	60	0.388	0.467	0.651	0.380	0.674	0.366	$f_{ca} = 0.388\sigma_3^{0.467}$	$f_{ca} = 0.651\sigma_3^{0.380}$	$f_{ca} = 0.674\sigma_3^{0.366}$

Chapter six

Conclusions and Recommendations

6.1 Introduction

This study dealt primarily with the characterization of asphalt mixtures for permanent deformation. In so doing, it considered five basic factors that affect permanent deformation of asphalt mixtures, which as a matter of fact were varied during the triaxial compression test. These factors were aggregates gradation, bitumen content, loading temperature, bitumen consistency and confining stress. A suitable model for the characterization of asphalt mixtures for permanent deformation was also sought. The conclusions include the concepts that the study settled at and agreed with. The recommendations include the use to which the researcher suggests may be made of the conclusions which includes further studies that may be carried out based on the findings of this research. The following conclusions and recommendations are based on the information gathered from the static triaxial compression test as guided by the objectives set in chapter one.

6.2 Conclusions

The development of comprehensive design methods of asphalt pavements which will help overcome the permanent deformation problem requires the determination of stress-strain behaviour and failure criteria of pavement materials. This must be ascertained at all possible loading and environmental conditions for all modes of failure. The determination of stress-strain behaviour of material is referred to as the physical characterization. Currently, the commonly used flexible pavement design methods are empirical. The methods use technological material characteristics rather than the fundamental materials behaviour. This fact requires an understanding of how various environmental and structural factors affect permanent deformation of asphalt mixtures. The influence of these factors on permanent deformation of asphalt mixtures must be investigated at all possible volumetric configurations of asphalt mixtures and loading temperatures.

Characterization of asphalt mixtures for permanent deformation may be carried out using triaxial compression test where five parameters may be used to characterize the asphalt mixtures for permanent deformation. These parameters are stress at initiation of dilation, stress at initiation of plasticity, tangent stiffness, maximum stress and poisson's ratio.

The conclusions reached in this study include the following;

1. Asphalt mixtures made of dense, well graded aggregate gradation blends 2, 3 and 4 exhibited high resistance towards permanent deformation as compared to the open or gap graded aggregate gradation blend 1. This is an indication that dense aggregate gradations are desirable to mitigate the effects of permanent deformation in asphalt concrete layers. Dense, well graded aggregate gradations have fewer voids and more contact points between aggregate particles than open or gap – graded mixtures.
2. Lower viscosity asphalts (those made with high penetration grade bitumen say 180/200), make asphalt mixtures less stiff and therefore more susceptible to permanent deformation especially at high temperatures; harder or more viscous (those made with low penetration grade bitumen say 60/70) asphalts are more stiff and hence best suited to resist high traffic loads at high temperatures.
3. The bitumen binder content influences the asphalt mixtures ability to resist permanent deformation. High bitumen contents increase permanent deformation potential. Binder contents are especially important when gap-graded aggregate gradations are used. Optimum bitumen content as determined by Marshall Mix design procedures is appropriate to mitigate the negative effects on permanent deformation. Degree of compaction is the primary quality parameter of the prepared test specimen especially when the mixture is critically designed which means it has low bitumen content to produce high resistance to permanent deformation.
4. Loading temperature has a significant effect on permanent deformation of asphalt mixtures. The permanent deformation of asphalt mixtures increases with increase in loading temperature. To come up with asphalt mixtures which are resistant to permanent deformation the temperatures used to design should be the highest possible to produce the most unfavorable pavement conditions.
5. The permanent deformation response of asphalt mixtures is highly dependent on the loading conditions. In particular the effect of confining stress on permanent deformation is very significant. The maximum stresses, the tangent stiffness and stress at initiation of plasticity

were all found to increase with increase in confining stresses. Thus it is necessary to find ways of estimating field confining stresses and to use this in laboratory testing of materials. The resistance of both gap–graded and well graded asphalt mixtures to permanent deformation at any given loading temperature is to a large extent dependent on the effect of confinement on the aggregate skeleton. The higher the confining stress, the higher the recorded maximum stress which translates to high resistance to permanent deformation.

6. Constitutive laws were identified as models which best describe permanent deformation of asphalt mixtures. Power and natural logarithmic laws were successfully used to model asphalt mixtures for permanent deformation. Power law of the form, $\mathrm{f_{ca}} = \mathrm{a}\sigma_3^{\mathrm{b}}$ was used to model the maximum stress with respect to the confining stress while natural logarithmic law of the form, $\mathrm{lnT_S} = \mathrm{C_1} + \mathrm{k_1 lnT}$ was used to model the tangent stiffness in respect to the loading temperature.

6.3 Recommendations

1. In order that the researcher is able to compare the laboratory measured values with the field measured values, it is recommended that the characterization of asphalt mixtures for permanent deformation should be conducted at a larger scale in the field, using a real pavement. This way it will be possible to get an insight on how to model the various parameters such as the confining stresses and the loading speeds.
2. In order to be able to capture a wide range of aggregates gradation it is recommended that the triaxial test should be conducted using a larger cylindrical specimen say 100mm diameter by 200mm long. The cylindrical specimens used in this study were 38mm diameter by 76mm long and only a narrow range of aggregate gradation could be accommodated (0/14).This range of aggregate gradation is only suitable for making asphalt mixtures suitable for wearing and binder courses of the typical Kenyan pavements. To be able to characterize bituminous mixtures for road base e.g. dense bitumen macadam (DBM) a large specimen size is required able to accommodate 0/40 aggregate gradation range.
3. In order to be able to simulate repeated wheel loads, it is recommended that the triaxial compression test in this study be repeated only that this time round a dynamic triaxial compression test should be used instead. A dynamic triaxial test will make it possible to simulate repeated wheel loads as opposed to static loading used in this study. Parameters such as creep and dynamic modulus can be obtained which will help to further characterize asphalt mixtures for permanent deformation. In characterization of permanent deformation of asphalt mixtures, loading time is very important and only the dynamic triaxial test is capable to simulate the loading time.
4. In order to be able to provide an insight into the distinct roles played by the aggregate skeleton and the bitumen binder components in an asphalt mixture, it is recommended that studies be conducted to help understand and characterize the separate contribution of the components in asphalt mixtures.

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APPENDIX 1

Marshall Mix Design Method

Various Equations are applicable as shown below;

Specific Gravity of Mixed Aggregates (S.G.M.A)

$$S.G.M.A = \frac{P_1+P_2+ \ldots\ldots + P_{n-1} + P_n}{P_1/G_1+P_2/G_2+ \ldots\ldots + P_{n-1}/G_{n-1}+P_n/G_n} \quad \ldots\ldots 1.1$$

Where;

P_1 = % weight of first aggregate fraction.

P_2 = % weight of second aggregate fraction

P_{n-1} = % weight of $(n-1)^{th}$ aggregate fraction

P_n = % weight of Nth aggregate fraction

G_1 = Specific gravity of first aggregate fraction

G_2 = Specific gravity of second aggregate fraction.

G_{n-1} = Specific gravity of $(n-1)^{th}$ aggregate fraction

G_n = Specific gravity of n^{th} aggregate fraction

Volume of Each Specimen (V) in cm^3

Is equal to the weight of displaced water in *g*

V = Weight of specimen in air – weight of specimen in water ……………………………………1.2

Compacted Density of Mix (C.D.M)

$$C.D.M = \frac{W_A}{W_A-W_W} \quad \ldots\ldots 1.3$$

Where;

W_A = Weight of specimen in air

W_W = Weight of specimen in water

The C.D.M of the mix is taken as the mean C.D.M value for three or four specimens.

Compacted Density of Mixed Aggregates (C.D.M.A)

$C.D.M.A = W_{CDM}/V_C$ ………………………………………………………………………1.4(a)

Or

$C.D.M.A = C.D.M/ (1+B/100)$ in g/cm^3 ……………………………………………..………1.4(b)

Where;

W_{CDM} = Weight of compacted mix aggregates

V_C = Volume of container

B = Percentage bitumen content

Where the container is Marshall mould or proctor mould.

Specific Gravity of Mix (S.G.M)

$$S.G.M = \frac{100+B}{100/S.G.M.A + B/S.G\text{ (of bitumen)}} \quad (g/cm^3) \qquad \text{……1.5}$$

Where S.G.M is required to control work.

Voids in Mix (V.I.M)

$$V.I.M = \frac{S.G.M - C.D.M}{S.G.M} \times 100\% \qquad \text{……1.6}$$

Voids in Mixed Aggregates (V.M.A)

$$V.M.A = \frac{S.G.M.A - C.D.M.A}{S.G.M.A} \times 100\% \qquad \text{……1.7}$$

Voids Filled With Bitumen (V.F.B)

$$V.F.B = \frac{V.M.A - V.I.M}{S.G.M.A} \times 100\% \qquad \text{……1.8}$$

Marshall Stiffness (M_S)

$MS = M_{ST}/M_{FL}$ (lb/0.01 inch or kg/mm) ……………………………………...………………1.9

Where;

M_{ST} = Marshall Stability in lbs/0.01 inch or kg/mm.

M_{FL} = Marshall Flow in 0.01 in or mm

Weight per M^2

Weight of the mix in kg/m^2 = C.D.M x layer thickness (t) in cm x 10 ..………………………… 1.10

M^2 per Tonne of Mix

M^2/Tonne = 1000/ (kg/m^2) ………………………..……………………………………………..1.11

TABLE : 1.1 : MIX DESIGN RESULTS											
AGGREGATE GRADING : 1											
BITUMEN PENETRATION GRADE : 60/70											
A	B	C	D	E	F	G	H	J	K	L	M
BITUMEN CONTENT	C.D.M	S.G.M	BITUMEN VOLUME	AGRREGATE VOLUME	V.M.A	V.I.M	V.F.B	UNIT WEIGHT	AVERAGE CORRECTED STABILITY	AVERAGE FLOW VALUE	AVERAGE MARSHALL STIFFNESS
%wt	g/cm3	g/cm3	%	%	%	%	%	lb/ft3	lb	0.01in	lb/0.01in
4	2.231	2.367	9.60	84.67	16.10	5.73	64.41	139.5	1598	11	145
5	2.246	2.346	10.34	82.93	15.20	4.28	71.84	140.6	1815	13	140
6	2.234	2.341	13.26	82.18	14.80	4.56	69.19	141.2	1898	17	112
7	2.218	2.315	14.73	81.09	16.30	4.18	74.36	140.5	1834	21	87
8	2.205	2.267	16.85	80.43	19.40	2.72	85.98	140.2	1707	27	63

TABLE : 1.2 : MIX DESIGN RESULTS											
AGGREGATE GRADING : 1											
BITUMEN PENETRATION GRADE : 80/100											
A	B	C	D	E	F	G	H	J	K	L	M
BITUMEN CONTENT	C.D.M	S.G.M	BITUMEN VOLUME	AGRREGATE VOLUME	V.M.A	V.I.M	V.F.B	UNIT WEIGHT	AVERAGE CORRECTED STABILITY	AVERAGE FLOW VALUE	AVERAGE MARSHALL STIFFNESS
%wt	g/cm3	g/cm3	%	%	%	%	%	lb/ft3	lb	0.01in	lb/0.01in
4	2.238	2.398	8.17	85.16	16.5	6.67	59.58	137.6	1961	13	151
5	2.223	2.337	10.63	84.49	15.9	4.88	69.31	138.7	1523	15	102
6	2.203	2.292	12.49	83.63	16.7	3.88	76.77	140.6	1406	17	83
7	2.198	2.264	14.56	82.51	17.2	2.93	82.97	140.3	1250	25	50
8	2.192	2.230	16.84	81.47	18.4	1.69	90.82	138.5	1395	27	52

TABLE : 1.3 : MIX DESIGN RESULTS											
AGGREGATE GRADING : 1											
BITUMEN PENETRATION GRADE : 180/200											
A	B	C	D	E	F	G	H	J	K	L	M
BITUMEN CONTENT	C.D.M	S.G.M	BITUMEN VOLUME	AGRREGATE VOLUME	V.M.A	V.I.M	V.F.B	UNIT WEIGHT	AVERAGE CORRECTED STABILITY	AVERAGE FLOW VALUE	AVERAGE MARSHALL STIFFNESS
%wt	g/cm3	g/cm3	%	%	%	%	%	lb/ft3	lb	0.01in	lb/0.01in
4	2.196	2.384	8.90	83.21	16.79	7.89	53.008	138.2	1350	12	113
5	2.210	2.358	9.98	83.76	16.24	6.26	61.453	139.8	1550	15	103
6	2.235	2.366	12.52	81.94	18.06	5.54	69.324	140.7	1795	16	112
7	2.203	2.295	14.93	81.07	18.93	4.00	78.87	139.1	1507	18	84
8	2.211	2.295	17.12	79.23	20.77	3.65	82.427	138.4	1289	21	61

TABLE : 1.4 : MIX DESIGN RESULTS											
AGGREGATE GRADING : 2											
BITUMEN PENETRATION GRADE : 60/70											
A	B	C	D	E	F	G	H	J	K	L	M
BITUMEN CONTENT	C.D.M	S.G.M	BITUMEN VOLUME	AGRREGATE VOLUME	V.M.A	V.I.M	V.F.B	UNIT WEIGHT	AVERAGE CORRECTED STABILITY	AVERAGE FLOW VALUE	AVERAGE MARSHALL STIFFNESS
%wt	g/cm3	g/cm3	%	%	%	%	%	lb/ft3	lb	0.01in	lb/0.01in
4	2.243	2.382	10.61	83.55	16.45	5.84	64.50	130	1250	15	83
5	2.261	2.350	11.96	84.26	15.74	3.78	75.98	138	2400	16	150
6	2.254	2.285	14.63	84.01	15.99	1.36	91.49	143	3150	19	166
7	2.235	2.261	16.24	82.61	17.39	1.15	93.39	140	3000	27	111
8	2.224	2.249	18.47	80.44	19.56	1.09	94.43	136	2670	30	89

TABLE : 1.5 : MIX DESIGN RESULTS											
AGGREGATE GRADING : 2											
BITUMEN PENETRATION GRADE : 80/100											
A	B	C	D	E	F	G	H	J	K	L	M
BITUMEN CONTENT	C.D.M	S.G.M	BITUMEN VOLUME	AGRREGATE VOLUME	V.M.A	V.I.M	V.F.B	UNIT WEIGHT	AVERAGE CORRECTED STABILITY	AVERAGE FLOW VALUE	AVERAGE MARSHALL STIFFNESS
%wt	g/cm3	g/cm3	%	%	%	%	%	lb/ft3	lb	0.01in	lb/0.01in
4	2.268	2.612	8.43	78.40	21.60	13.17	39.03	141.3	2704	13	208
5	2.244	2.440	10.36	81.60	18.40	8.04	56.30	140.2	2433	15	162
6	2.259	2.368	12.51	82.90	17.10	4.59	73.16	142.4	2260	17	133
7	2.225	2.305	14.72	81.80	18.20	3.48	80.88	137.6	1503	23	65
8	2.269	2.369	16.48	79.30	20.70	4.22	79.61	135.8	1042	27	39

TABLE : 1.6 : MIX DESIGN RESULTS											
AGGREGATE GRADING : 2											
BITUMEN PENETRATION GRADE : 180/200											
A	B	C	D	E	F	G	H	J	K	L	M
BITUMEN CONTENT	C.D.M	S.G.M	BITUMEN VOLUME	AGRREGATE VOLUME	V.M.A	V.I.M	V.F.B	UNIT WEIGHT	AVERAGE CORRECTED STABILITY	AVERAGE FLOW VALUE	AVERAGE MARSHALL STIFFNESS
%wt	g/cm3	g/cm3	%	%	%	%	%	lb/ft3	lb	0.01in	lb/0.01in
4	2.234	2.478	8.66	81.50	18.50	9.84	46.81	140.6	1201	16	75
5	2.215	2.366	10.32	83.30	16.70	6.38	61.80	139.1	1820	14	130
6	2.275	2.344	12.84	84.20	15.80	2.96	81.27	142.6	2200	20	110
7	2.238	2.271	14.76	83.80	16.20	1.44	91.11	139.7	2095	26	81
8	2.244	2.276	16.98	81.60	18.40	1.42	92.28	140.1	1606	23	70

TABLE : 1.7 : MIX DESIGN RESULTS											
AGGREGATE GRADING : 3											
BITUMEN PENETRATION GRADE : 60/70											
A	B	C	D	E	F	G	H	J	K	L	M
BITUMEN CONTENT	C.D.M	S.G.M	BITUMEN VOLUME	AGRREGATE VOLUME	V.M.A	V.I.M	V.F.B	UNIT WEIGHT	AVERAGE CORRECTED STABILITY	AVERAGE FLOW VALUE	AVERAGE MARSHALL STIFFNESS
%wt	g/cm3	g/cm3	%	%	%	%	%	lb/ft3	lb	0.01in	lb/0.01in
4	2.203	2.418	9.05	82.05	17.95	8.90	50.42	140.2	1805	14	129
5	2.215	2.374	11.33	81.97	18.03	6.70	62.84	141.6	1723	12	144
6	2.183	2.318	13.65	80.54	19.46	5.81	70.14	139.5	1685	13	130
7	2.191	2.301	15.22	79.98	20.02	4.80	76.02	139.8	1398	17	82
8	2.163	2.278	17.02	77.94	22.06	5.04	77.15	136.7	1204	22	55

TABLE : 1.8 : MIX DESIGN RESULTS											
AGGREGATE GRADING : 3											
BITUMEN PENETRATION GRADE : 80/100											
A	B	C	D	E	F	G	H	J	K	L	M
BITUMEN CONTENT	C.D.M	S.G.M	BITUMEN VOLUME	AGRREGATE VOLUME	V.M.A	V.I.M	V.F.B	UNIT WEIGHT	AVERAGE CORRECTED STABILITY	AVERAGE FLOW VALUE	AVERAGE MARSHALL STIFFNESS
%wt	g/cm3	g/cm3	%	%	%	%	%	lb/ft3	lb	0.01in	lb/0.01in
4	2.205	2.396	8.98	83.04	16.96	7.98	52.95	138.5	1888	14	135
5	2.219	2.381	10.43	82.76	17.24	6.81	60.50	139.1	1904	17	112
6	2.198	2.347	12.31	81.33	18.67	6.36	65.93	138.2	1306	18	73
7	2.163	2.275	14.65	80.43	19.57	4.92	74.86	137.1	1210	20	61
8	2.143	2.237	16.37	79.44	20.56	4.19	79.62	136.7	1104	21	53

TABLE : 1.9 : MIX DESIGN RESULTS											
AGGREGATE GRADING : 3											
BITUMEN PENETRATION GRADE : 180/200											
A	B	C	D	E	F	G	H	J	K	L	M
BITUMEN CONTENT	C.D.M	S.G.M	BITUMEN VOLUME	AGRREGATE VOLUME	V.M.A	V.I.M	V.F.B	UNIT WEIGHT	AVERAGE CORRECTED STABILITY	AVERAGE FLOW VALUE	AVERAGE MARSHALL STIFFNESS
%wt	g/cm3	g/cm3	%	%	%	%	%	lb/ft3	lb	0.01in	lb/0.01in
4	2.213	2.518	8.70	79.2	20.80	12.10	41.83	132.0	980	14	70
5	2.246	2.492	10.23	79.9	20.10	9.87	50.90	133.0	1427	12	119
6	2.187	2.353	12.45	80.5	19.50	7.05	63.85	135.0	1558	13	120
7	2.143	2.271	14.66	79.7	20.30	5.64	72.22	134.2	1205	15	80
8	2.021	2.105	16.59	79.4	20.60	4.01	80.53	133.8	1106	22	50

TABLE : 1.10 : MIX DESIGN RESULTS											
AGGREGATE GRADING : 4											
BITUMEN PENETRATION GRADE : 60/70											
A	B	C	D	E	F	G	H	J	K	L	M
BITUMEN CONTENT	C.D.M	S.G.M	BITUMEN VOLUME	AGRREGATE VOLUME	V.M.A	V.I.M	V.F.B	UNIT WEIGHT	AVERAGE CORRECTED STABILITY	AVERAGE FLOW VALUE	AVERAGE MARSHALL STIFFNESS
%wt	g/cm3	g/cm3	%	%	%	%	%	lb/ft3	lb	0.01in	lb/0.01in
4	2.301	2.517	8.75	82.68	17.32	8.57	50.52	143.2	1003	13	77
5	2.275	2.400	10.63	84.18	15.82	5.19	67.19	145.5	1144	15	76
6	2.253	2.358	12.35	83.19	16.81	4.46	73.47	142.8	2508	17	148
7	2.230	2.305	14.36	82.37	17.63	3.27	81.45	140.6	1961	28	70
8	2.215	2.290	16.64	80.07	19.93	3.29	83.49	139.7	1387	29	48

TABLE : 1.11 : MIX DESIGN RESULTS											
AGGREGATE GRADING : 4											
BITUMEN PENETRATION GRADE : 80/100											
A	B	C	D	E	F	G	H	J	K	L	M
BITUMEN CONTENT	C.D.M	S.G.M	BITUMEN VOLUME	AGRREGATE VOLUME	V.M.A	V.I.M	V.F.B	UNIT WEIGHT	AVERAGE CORRECTED STABILITY	AVERAGE FLOW VALUE	AVERAGE MARSHALL STIFFNESS
%wt	g/cm3	g/cm3	%	%	%	%	%	lb/ft3	lb	0.01in	lb/0.01in
4	2.306	2.488	8.29	84.39	15.61	7.32	53.11	143.1	1056	12	88
5	2.267	2.393	10.73	84.01	15.99	5.26	67.10	142.8	1684	17	99
6	2.245	2.347	12.56	83.11	16.89	4.33	74.36	141.5	2495	22	113
7	2.217	2.298	14.38	82.09	17.91	3.53	80.29	140.3	2206	31	71
8	2.189	2.249	16.89	80.43	19.57	2.68	86.31	139.7	1209	37	33

TABLE : 1.12 : MIX DESIGN RESULTS											
AGGREGATE GRADING : 4											
BITUMEN PENETRATION GRADE : 180/200											
A	B	C	D	E	F	G	H	J	K	L	M
BITUMEN CONTENT	C.D.M	S.G.M	BITUMEN VOLUME	AGRREGATE VOLUME	V.M.A	V.I.M	V.F.B	UNIT WEIGHT	AVERAGE CORRECTED STABILITY	AVERAGE FLOW VALUE	AVERAGE MARSHALL STIFFNESS
%wt	g/cm3	g/cm3	%	%	%	%	%	lb/ft3	lb	0.01in	lb/0.01in
4	2.294	2.421	8.73	86.01	13.99	5.26	62.40	143.7	756	13	58
5	2.277	2.384	10.38	85.13	14.87	4.49	69.80	144.1	1307	15	87
6	2.236	2.332	12.55	83.33	16.67	4.12	75.28	141.5	2608	23	113
7	2.211	2.282	14.81	82.09	17.91	3.10	82.69	140.1	2109	42	50
8	2.185	2.234	16.34	81.45	18.55	2.21	88.09	139.3	950	41	23

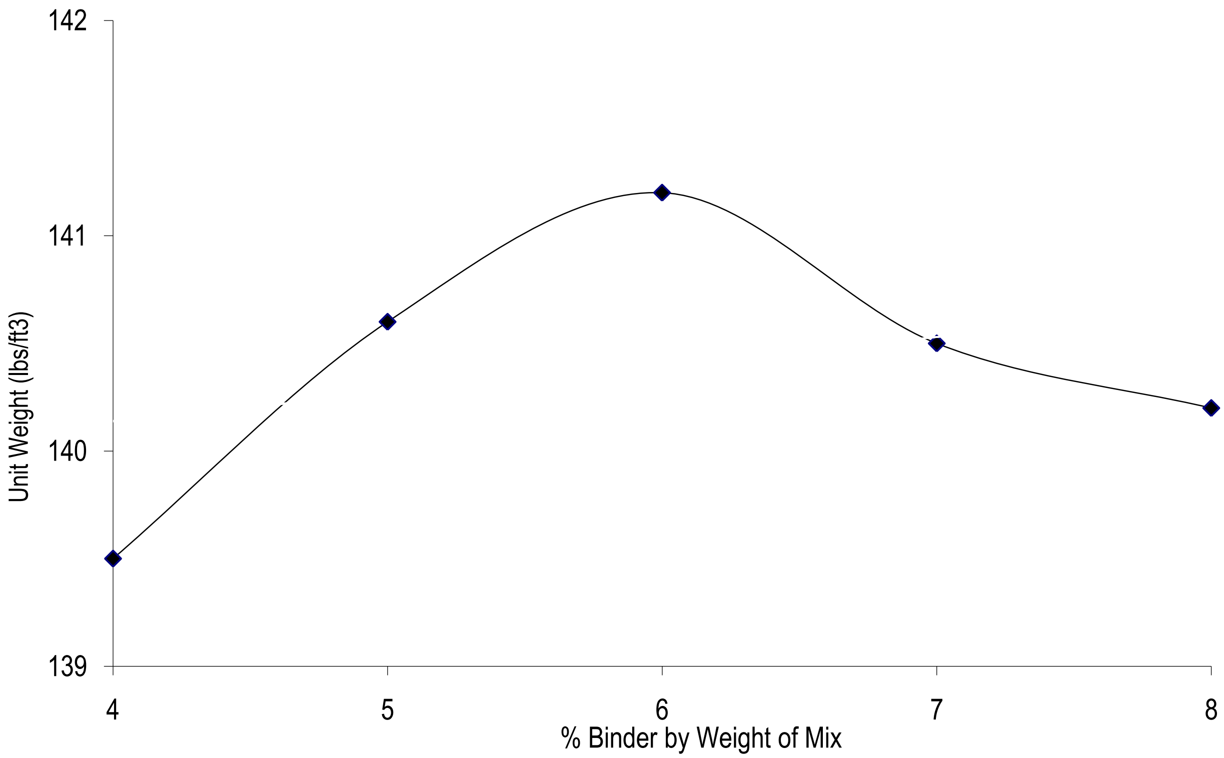

Figure 1.1.1 ; Unit Weight Vs % Bitumen Content

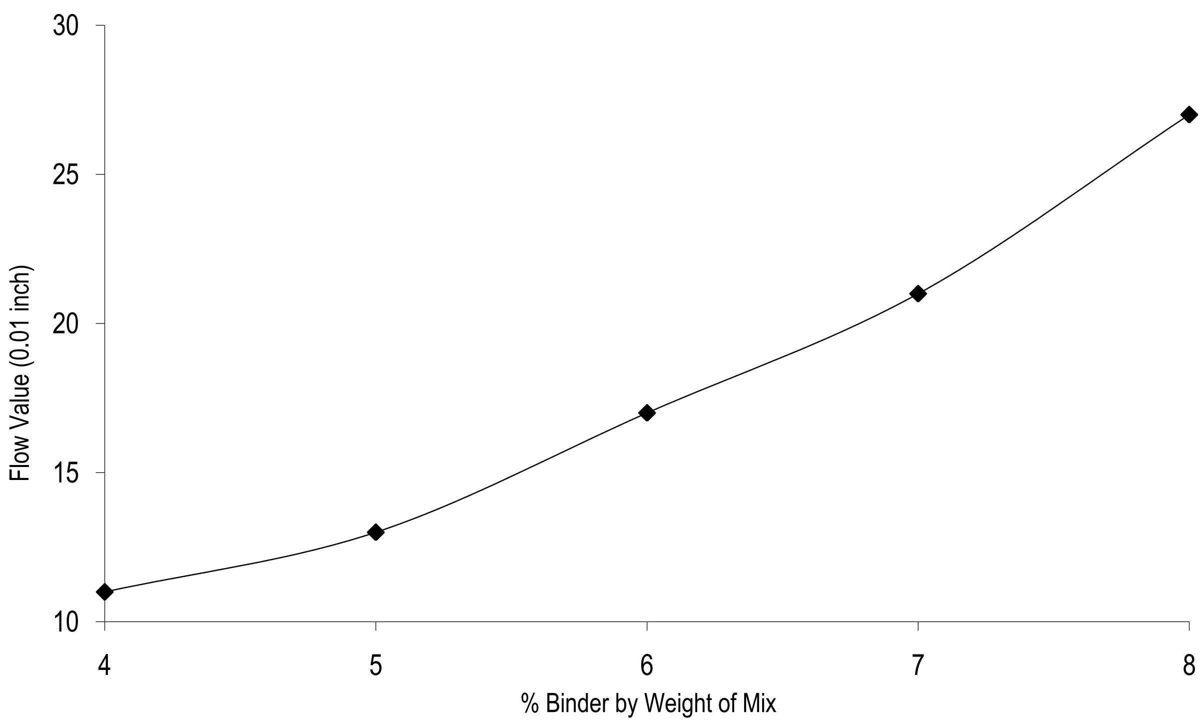

Figure 1.1.2 ; Flow Value Vs % Bitumen Content

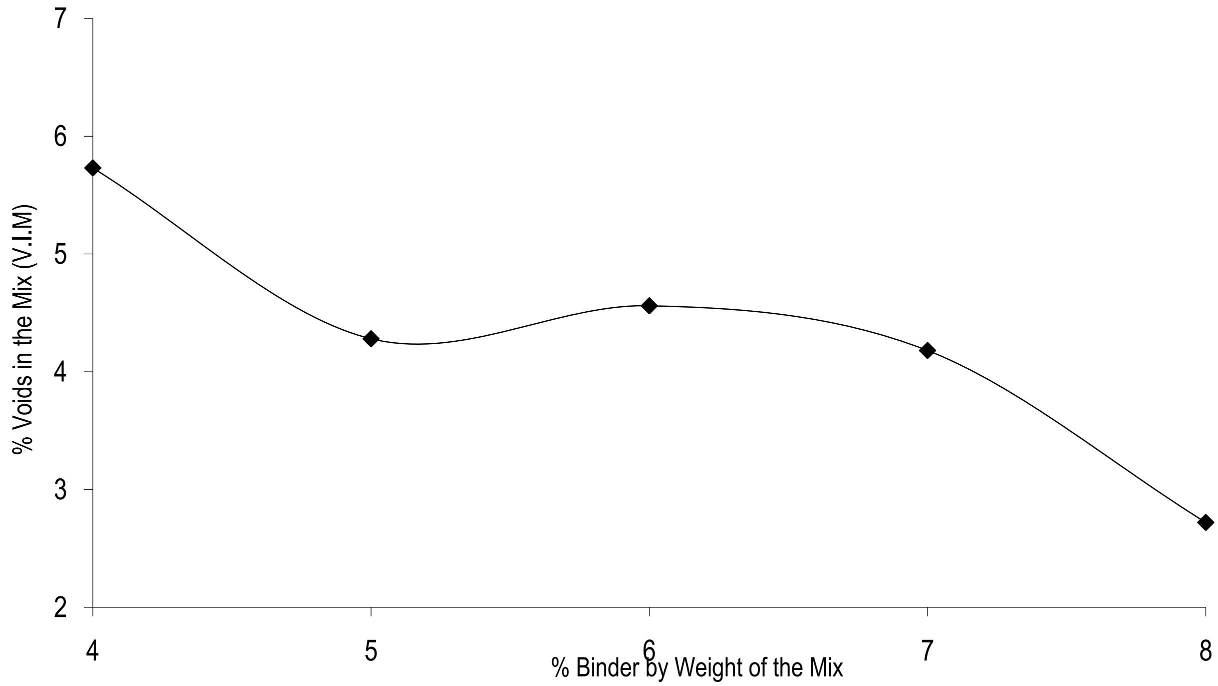

Figure 1.1.3 ; Voids in the Mix Vs % Bitumen Content

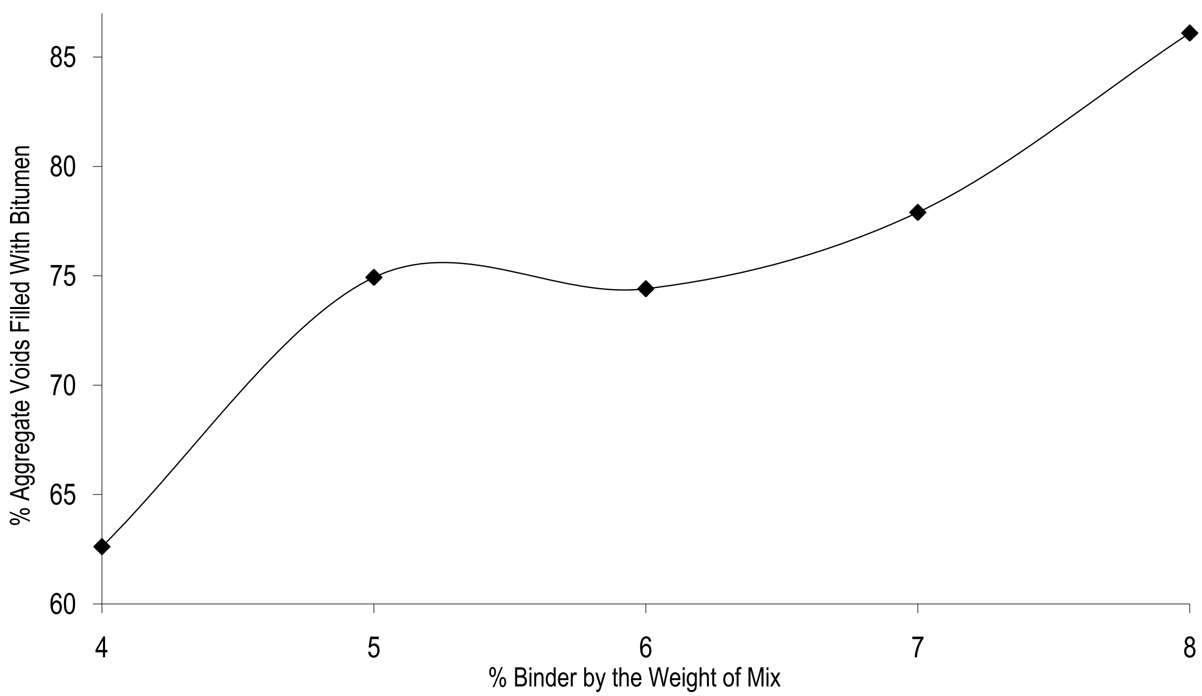

Figure 1.1.4 ; Voids Filled With Bitumen Vs % Bitumen Content

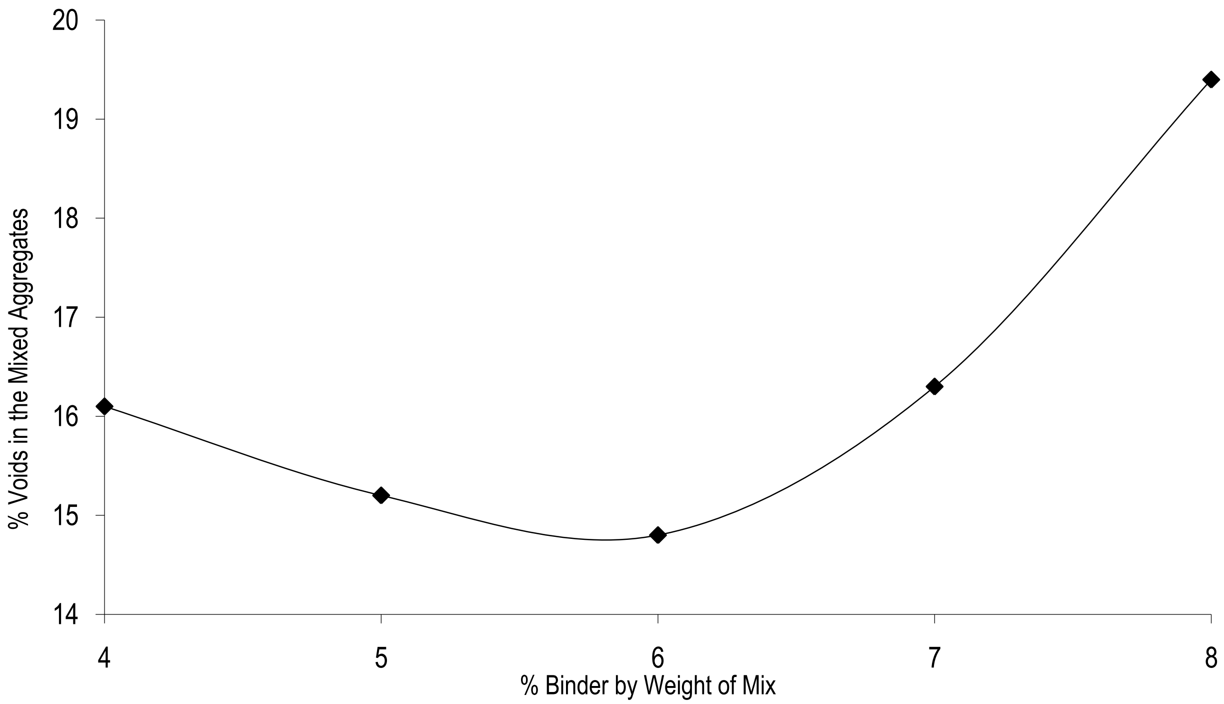

Figure 1.1.5 ; Voids in Mixed Aggregates Vs % Bitumen Content

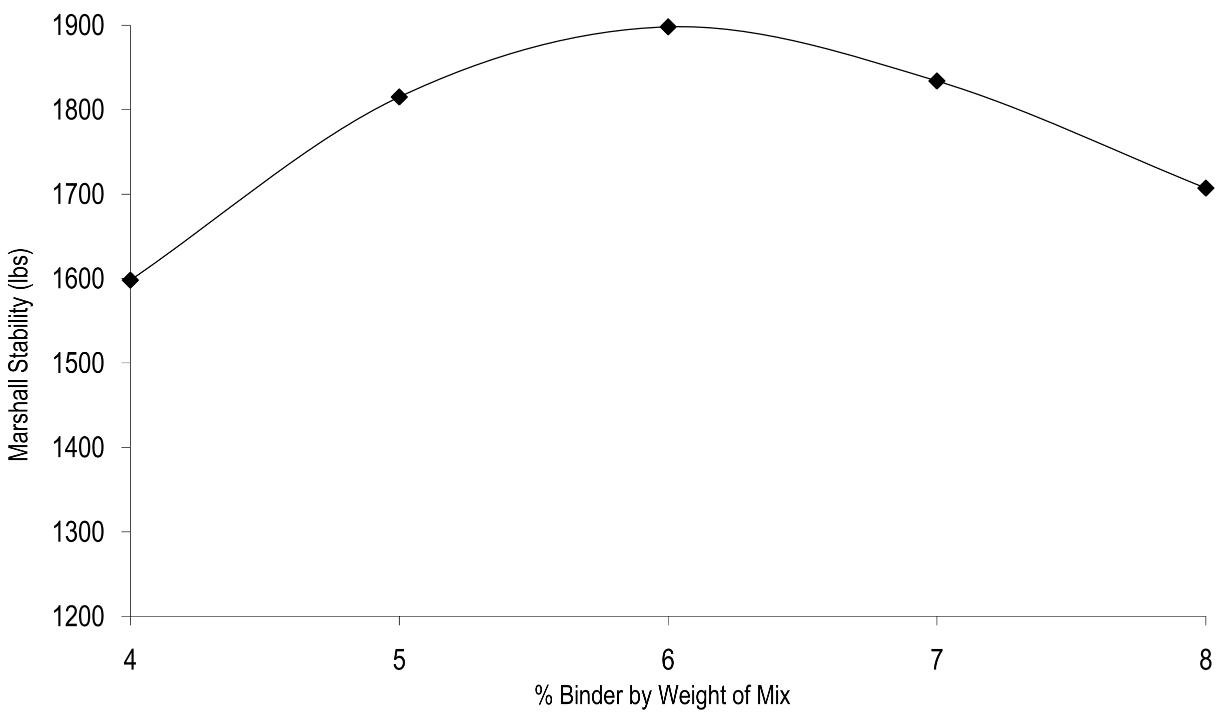

Figure 1.1.6 ; Marshall Stability Vs % Bitumen Content

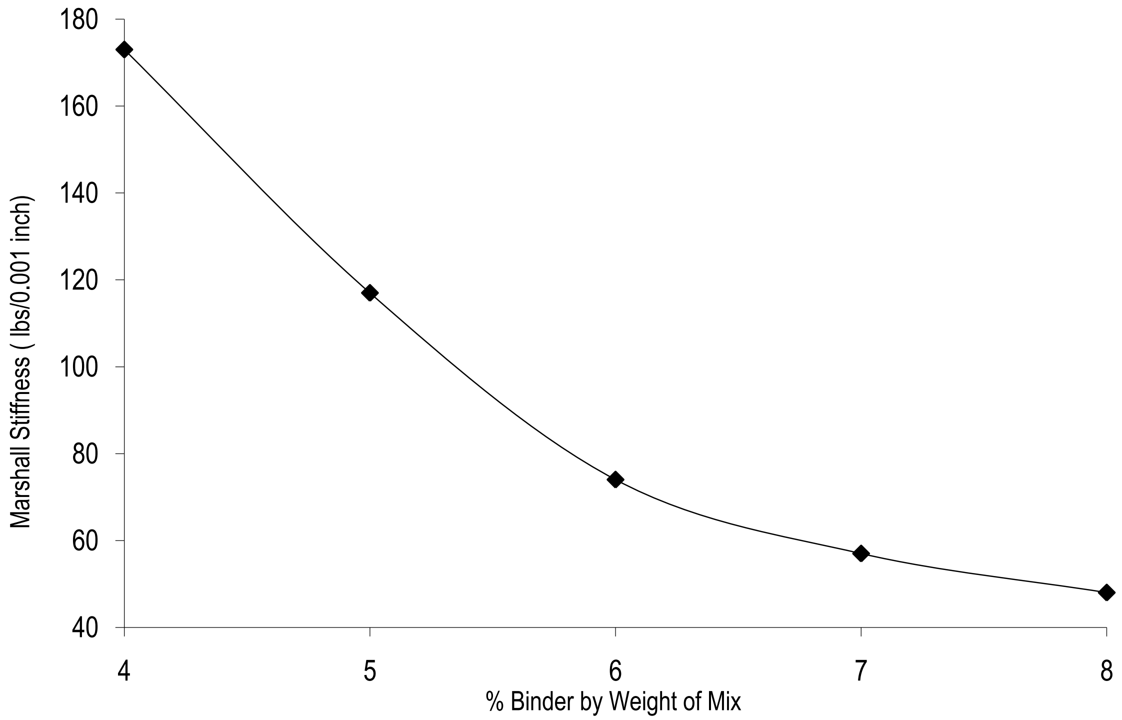

Figure 1.1.7 ; Marshall Stiffness Vs % Bitumen Content

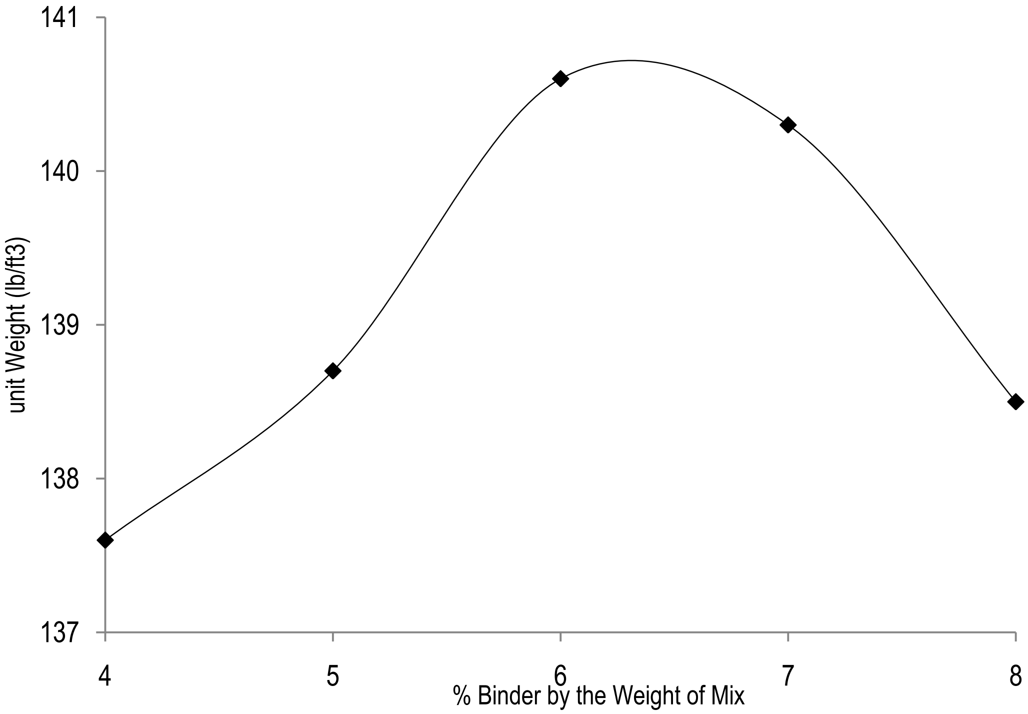

Figure 1.2.1 ; Unit Weight of the Mix Vs % Bitumen Content

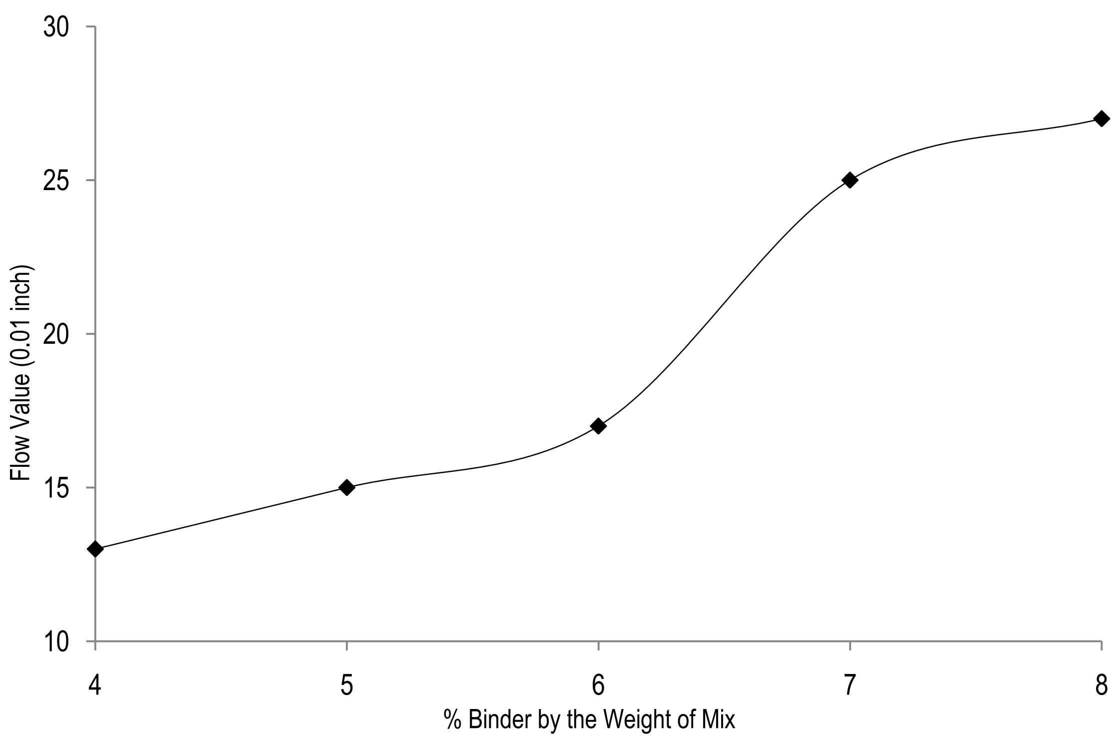

Fig 1.2.2 ; Flow Value Vs % Bitumen Content

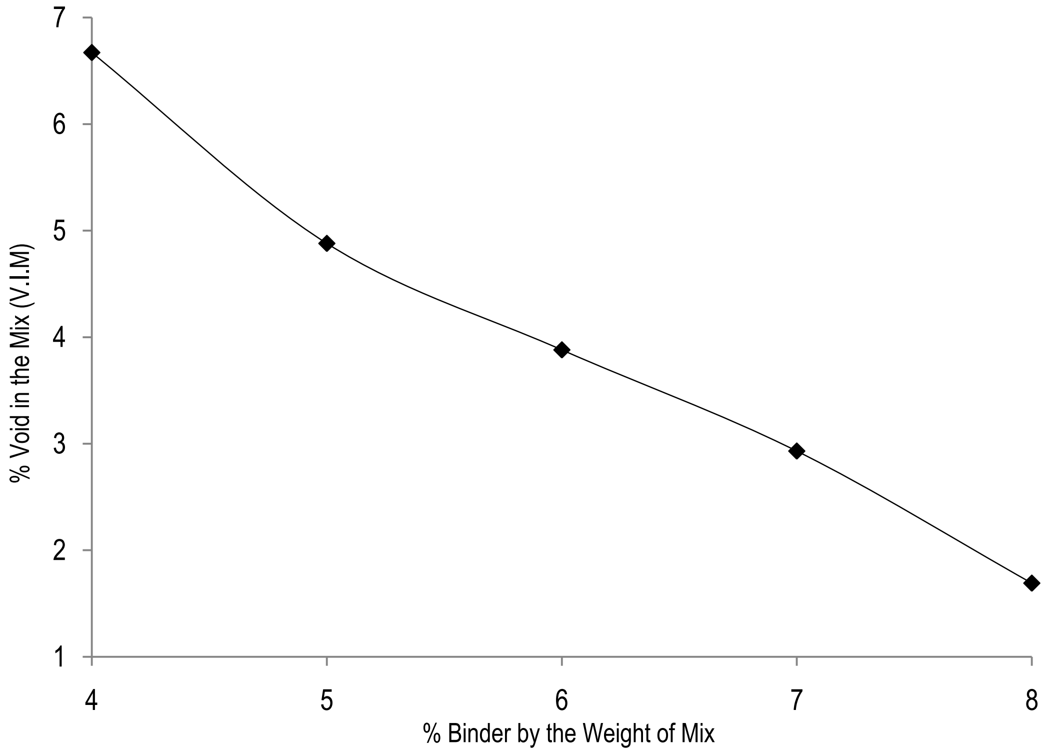

Fig 1.2.3 ; % Void in the Mix Vs % Bitumen Content

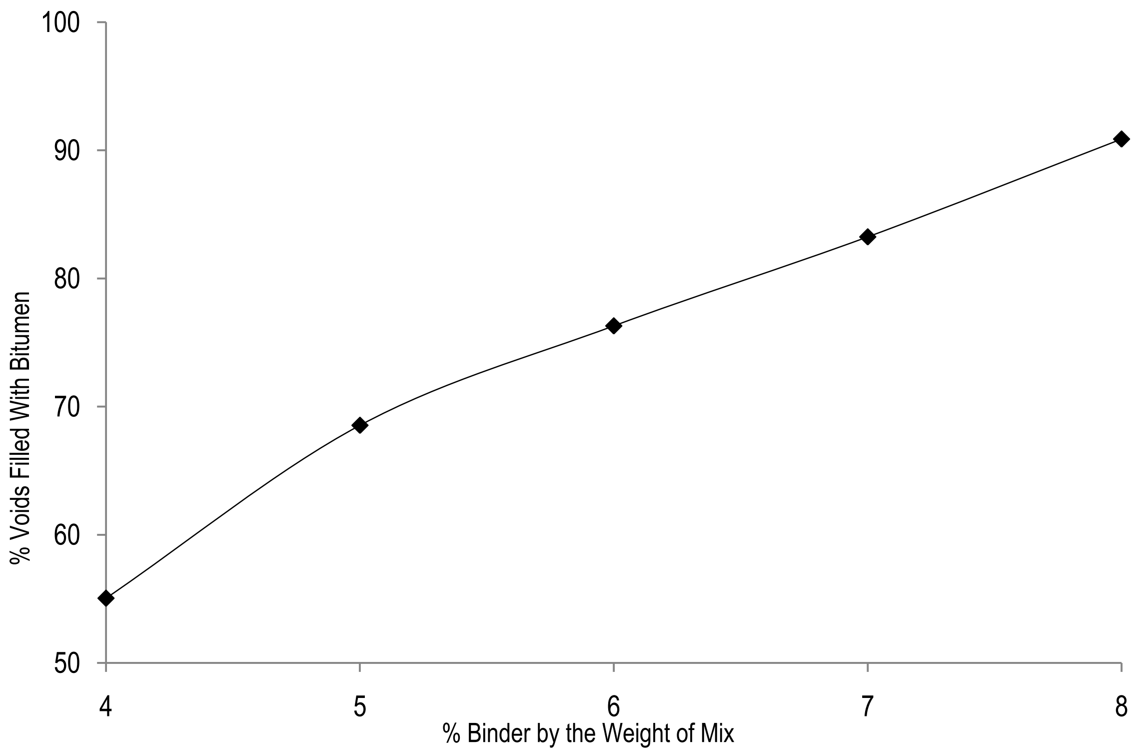

Fig 1.2.4 ; % Aggregate Voids Filled With Bitumen Vs % Bitumen Content

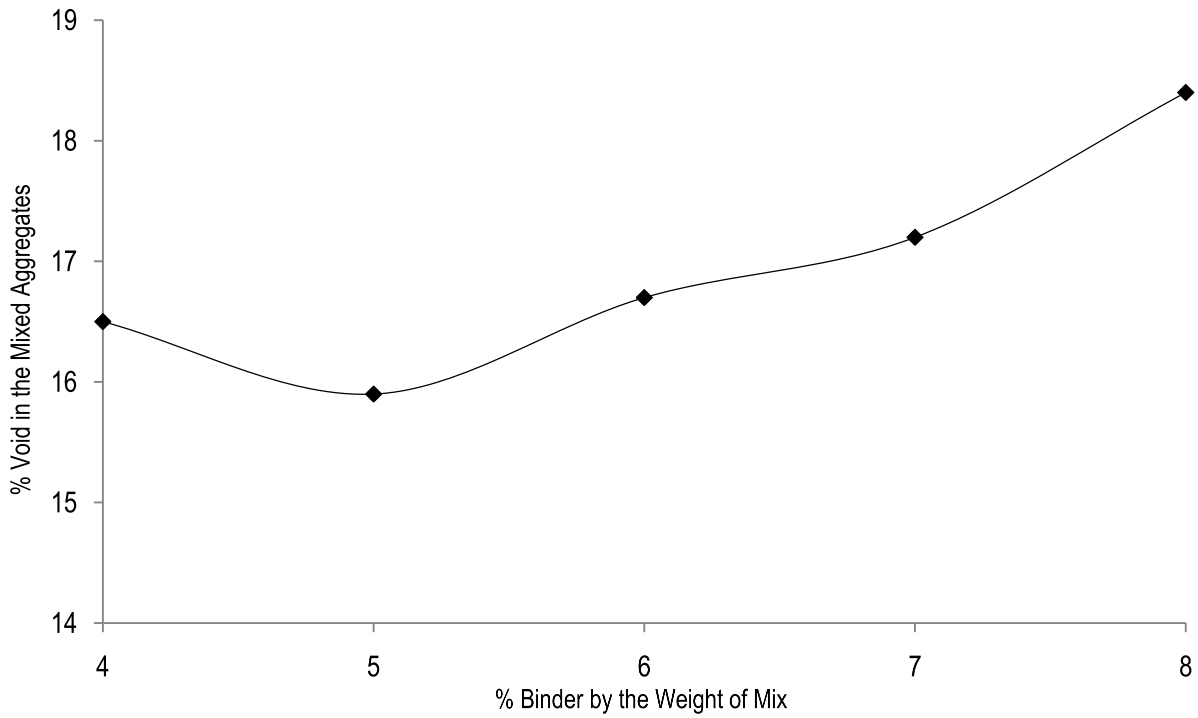

Fig 1.2.5 ; % Void in the Mixed Aggregates Vs % Bitumen Content

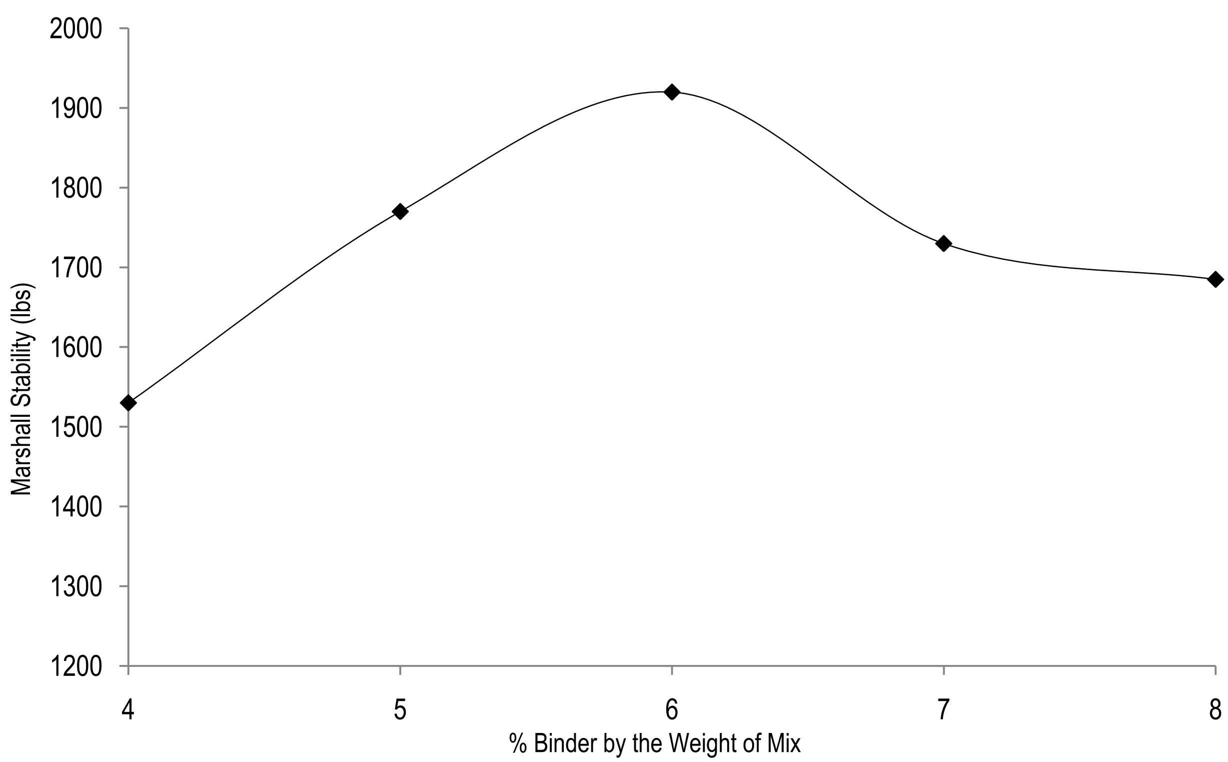

Fig 1.2.6. Marshall Stability Vs % Bitumen Content

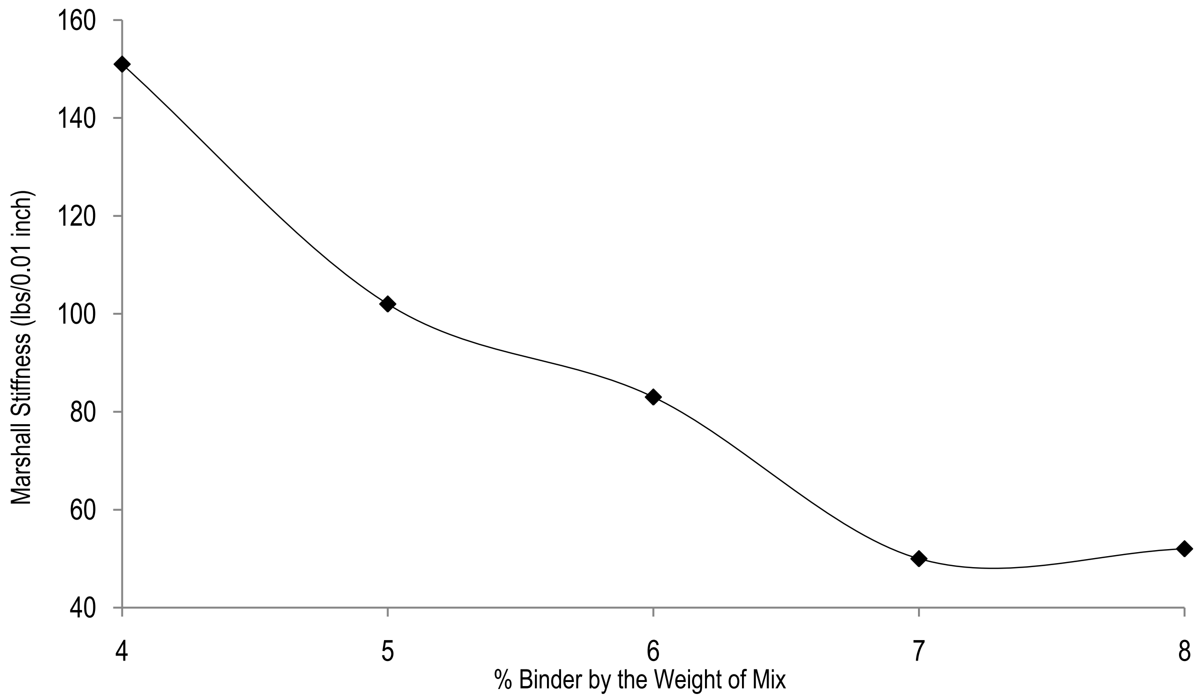

Fig 1.2.7 ; Marshall Stiffness Vs % Bitumen Content

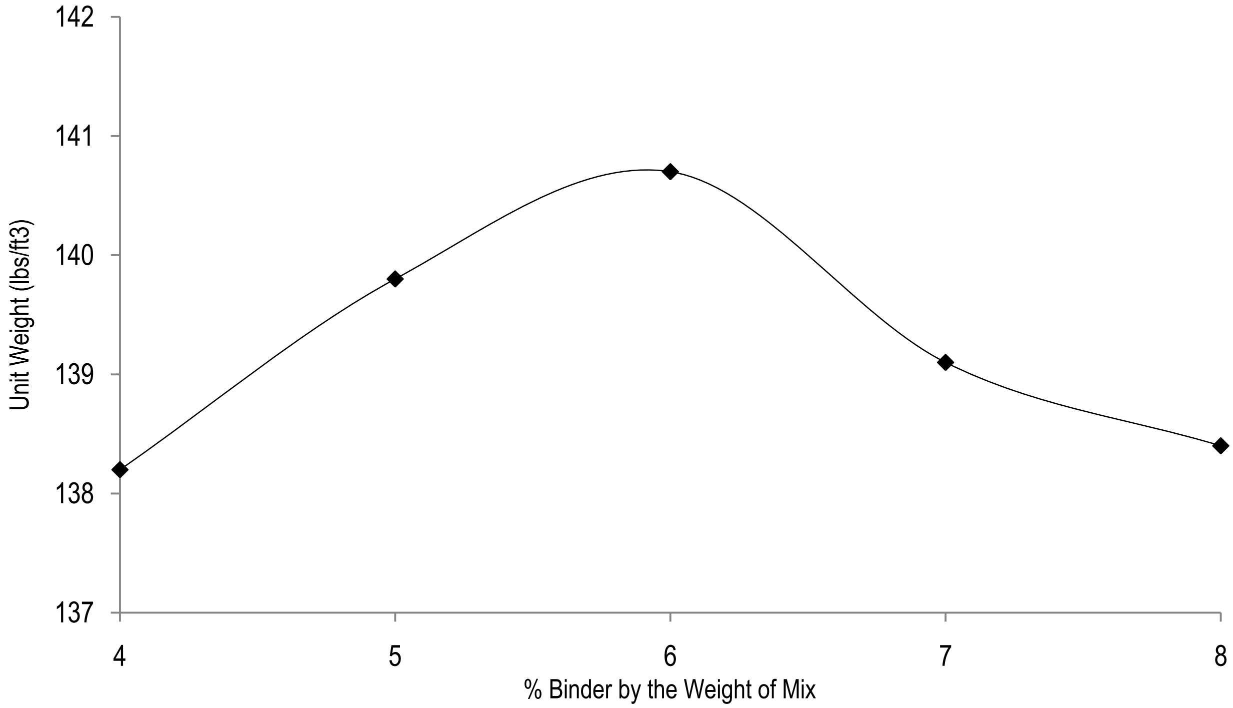

Fig 1.3.1 ; Unit Weight Vs % Bitumen Content

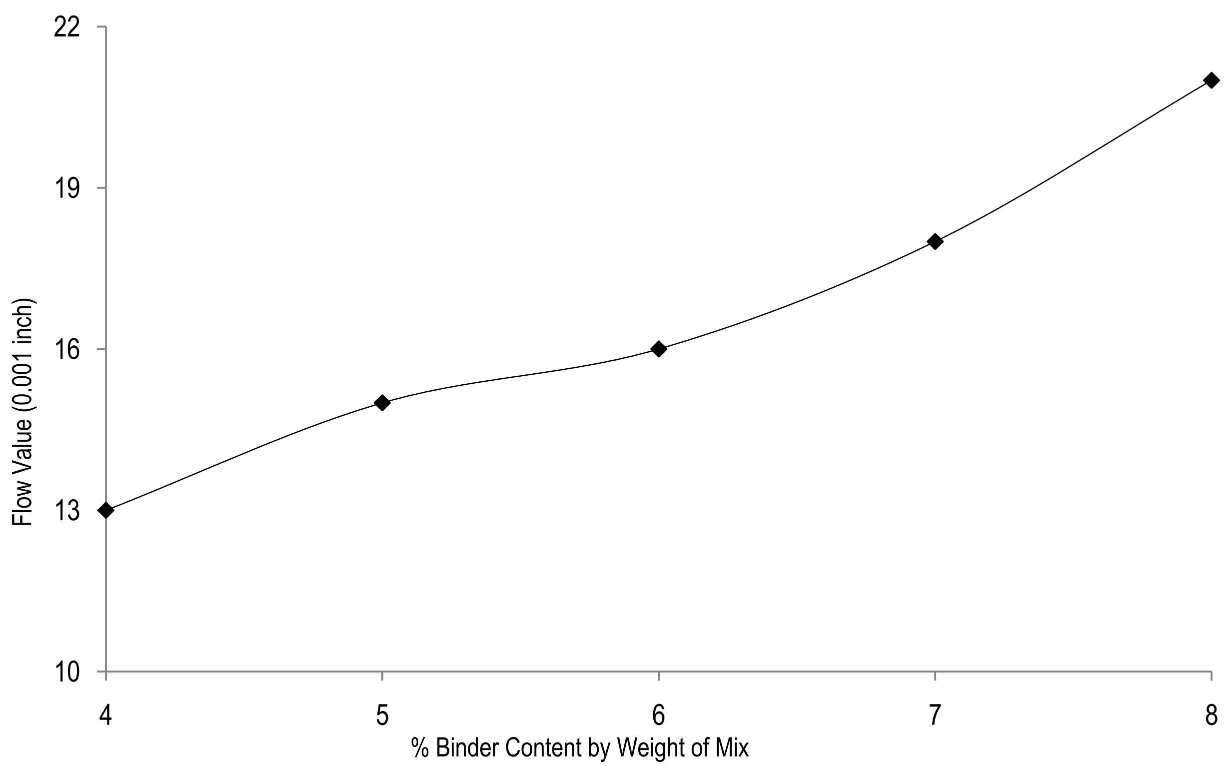

Fig 1.3.2 ; Flow Value Vs % Bitumen Content

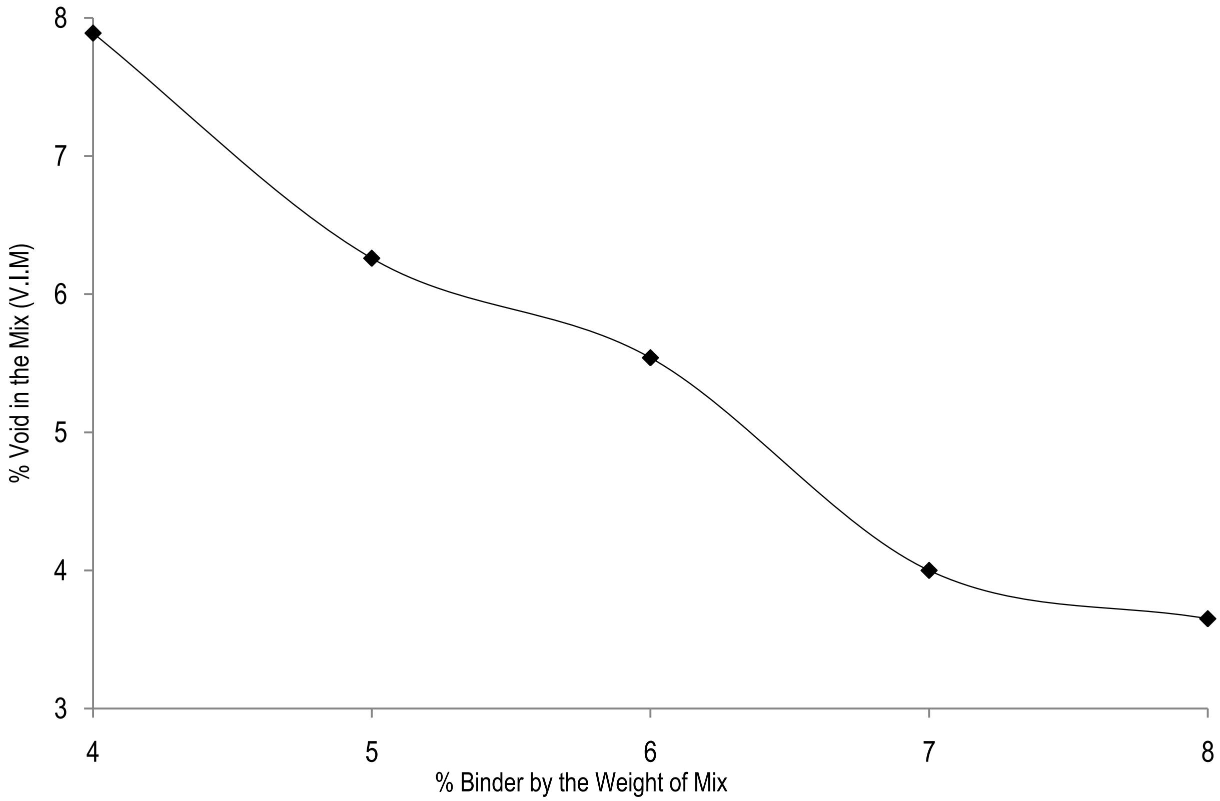

Fig 1.3.3 ; % Voids in the Mix Vs % Bitumen Content

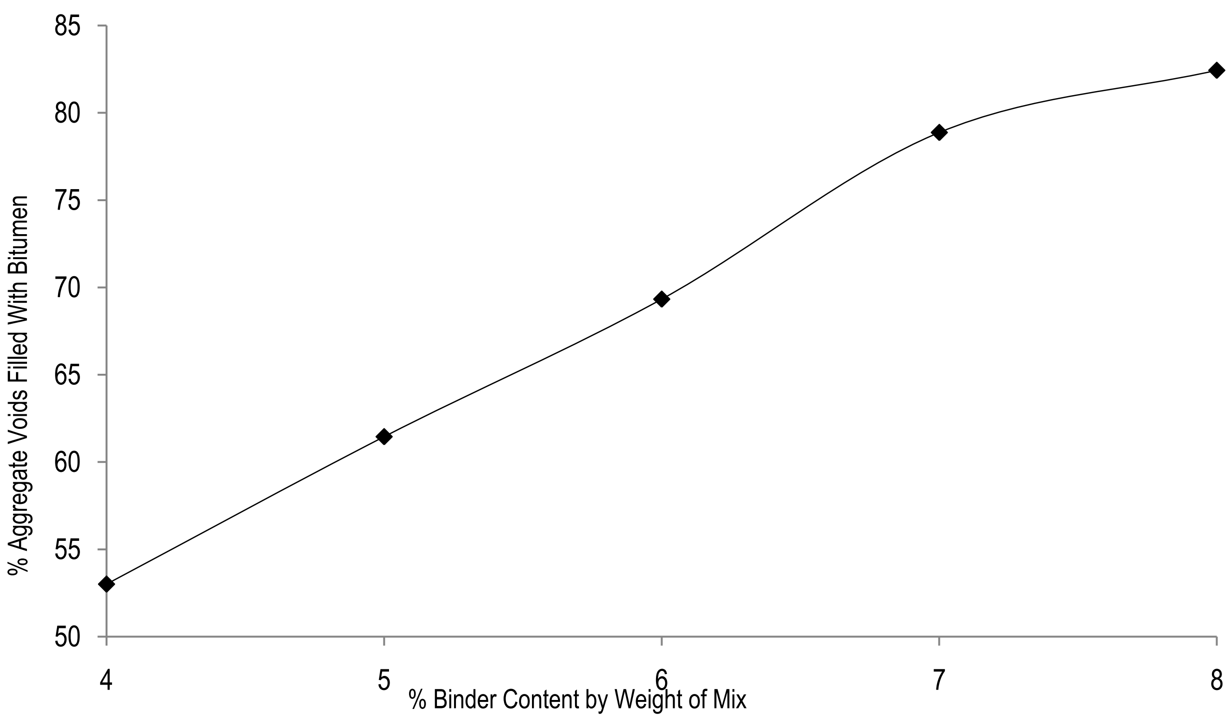

Fig 1.3.4 ; % Aggregate Voids Filled With Bitumen Vs % Bitumen Content

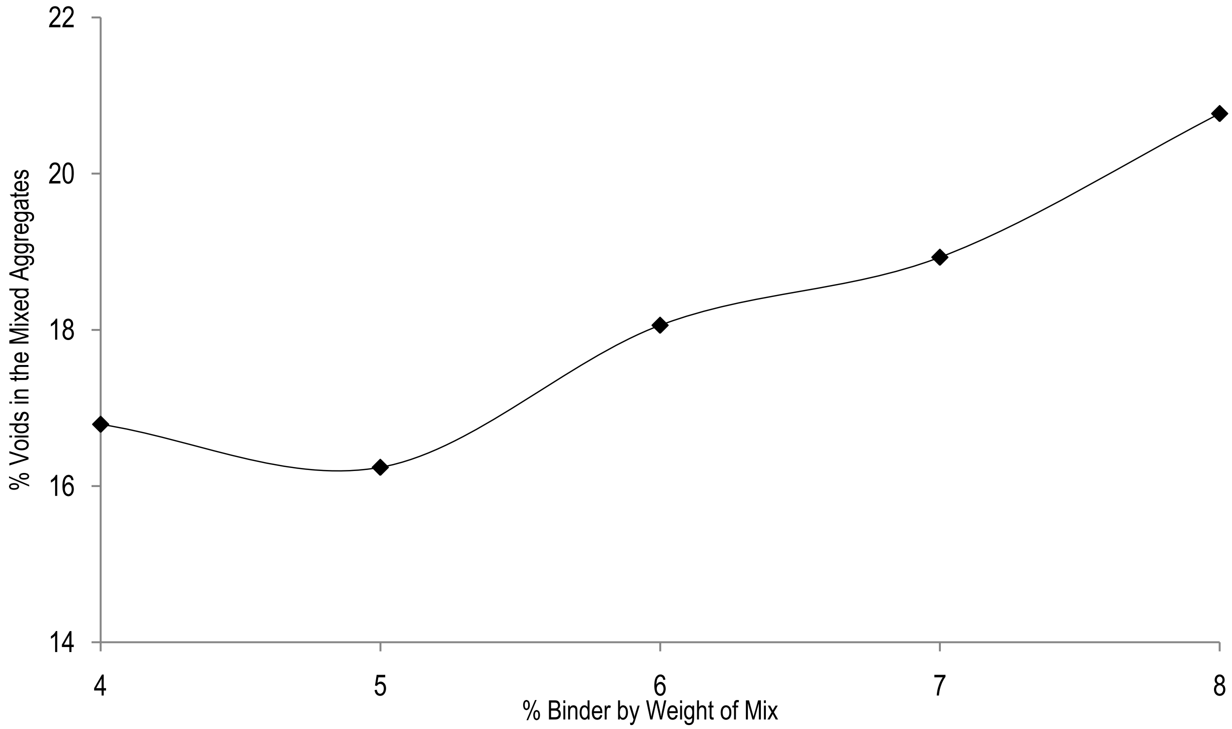

Fig 1.3.5 ; % Void in the Mixed Aggregates Vs % Bitumen Content

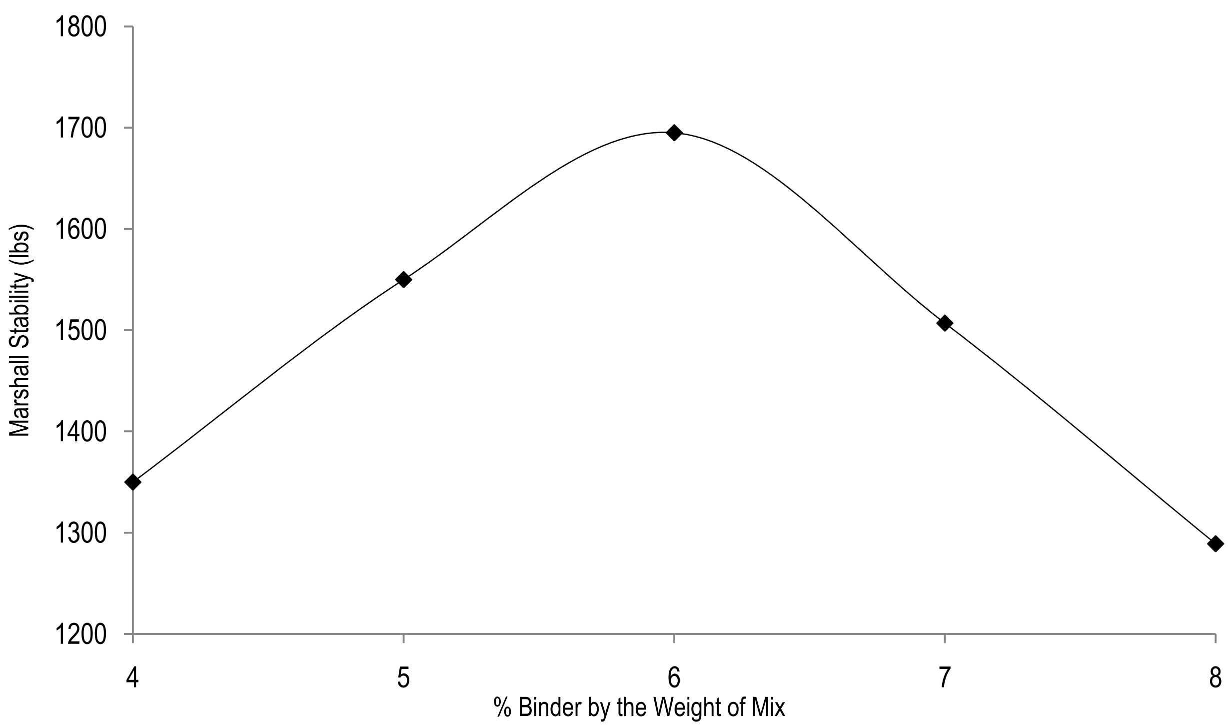

Fig 1.3.6 ; Marshall Stiffness Vs % Bitumen Content

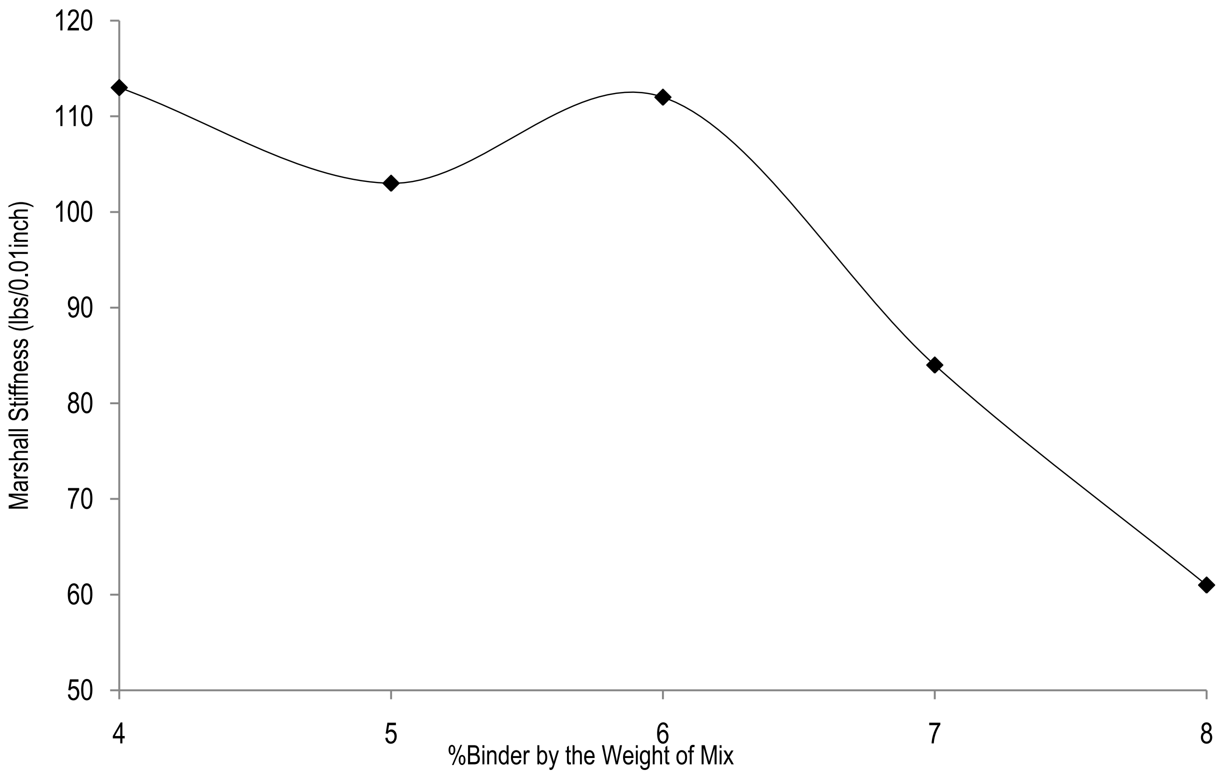

Fig 1.3.7 ; Marshall Stiffness Vs % Bitumen Content

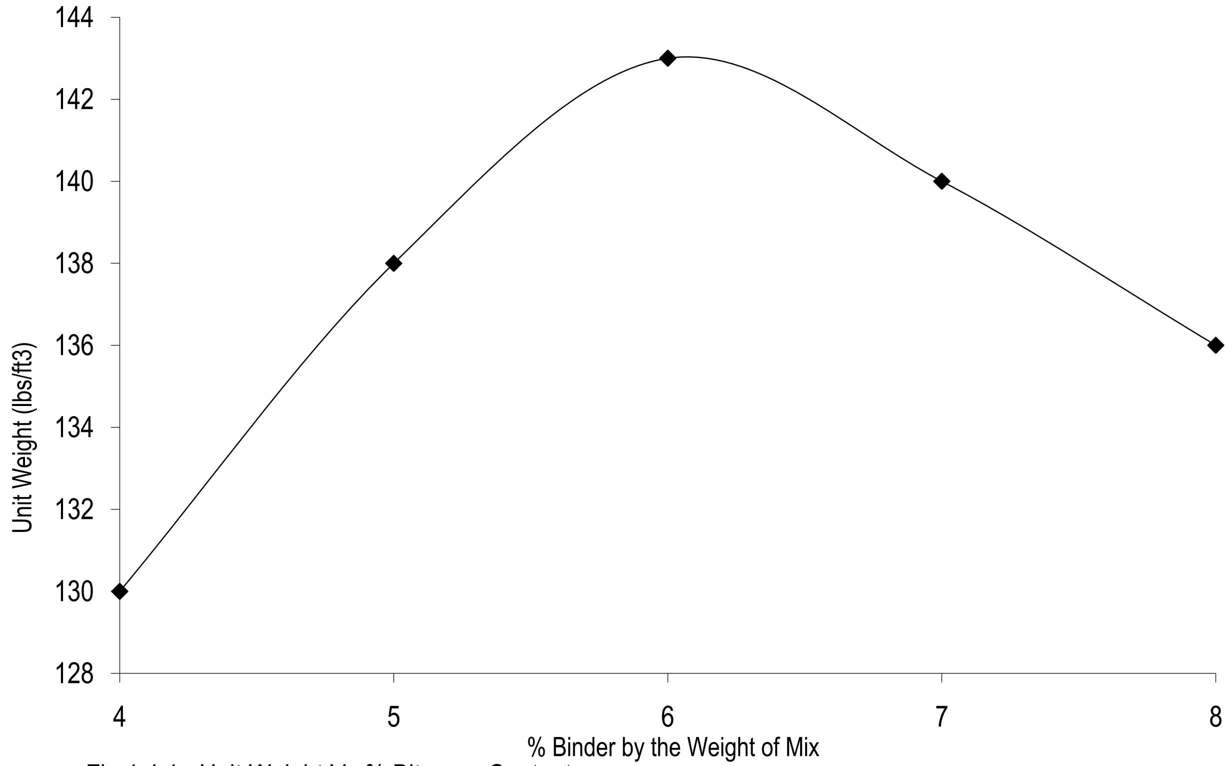

Fig 1.4.1 ; Unit Weight Vs % Bitumen Content

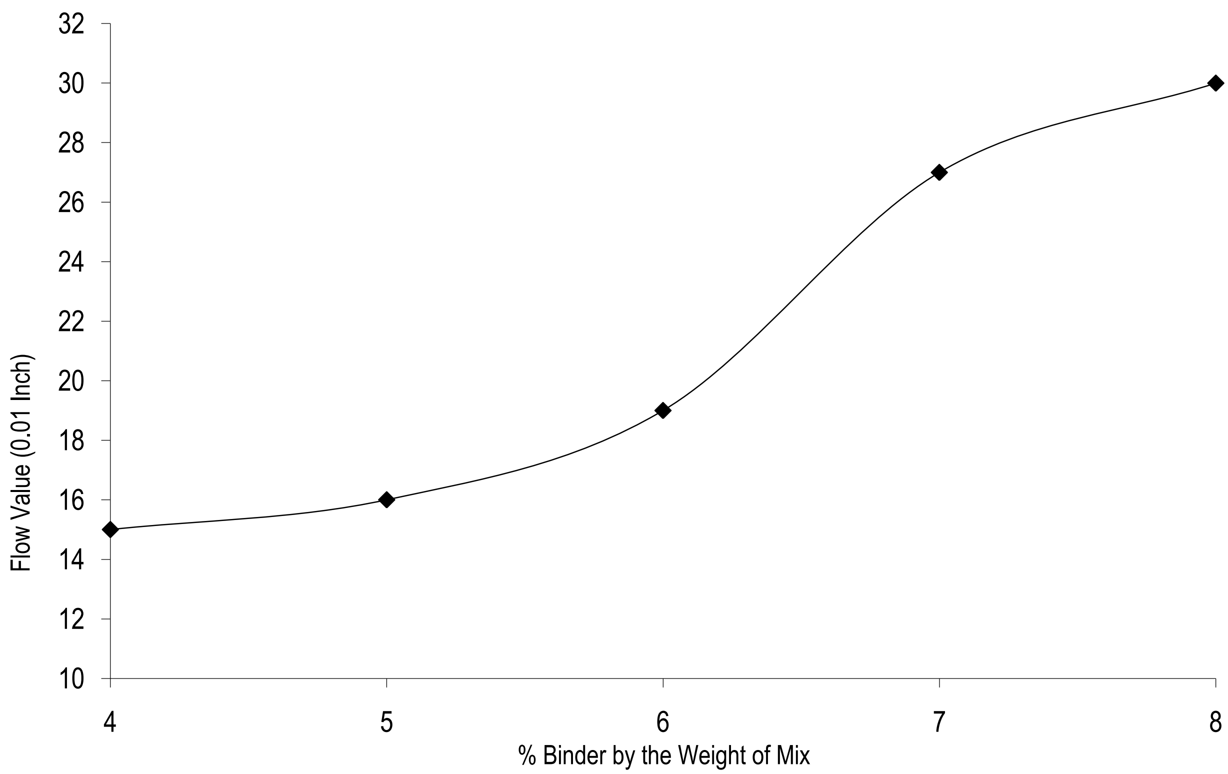

Fig 1.4.2 ; Flow Value Vs % Bitumen Content

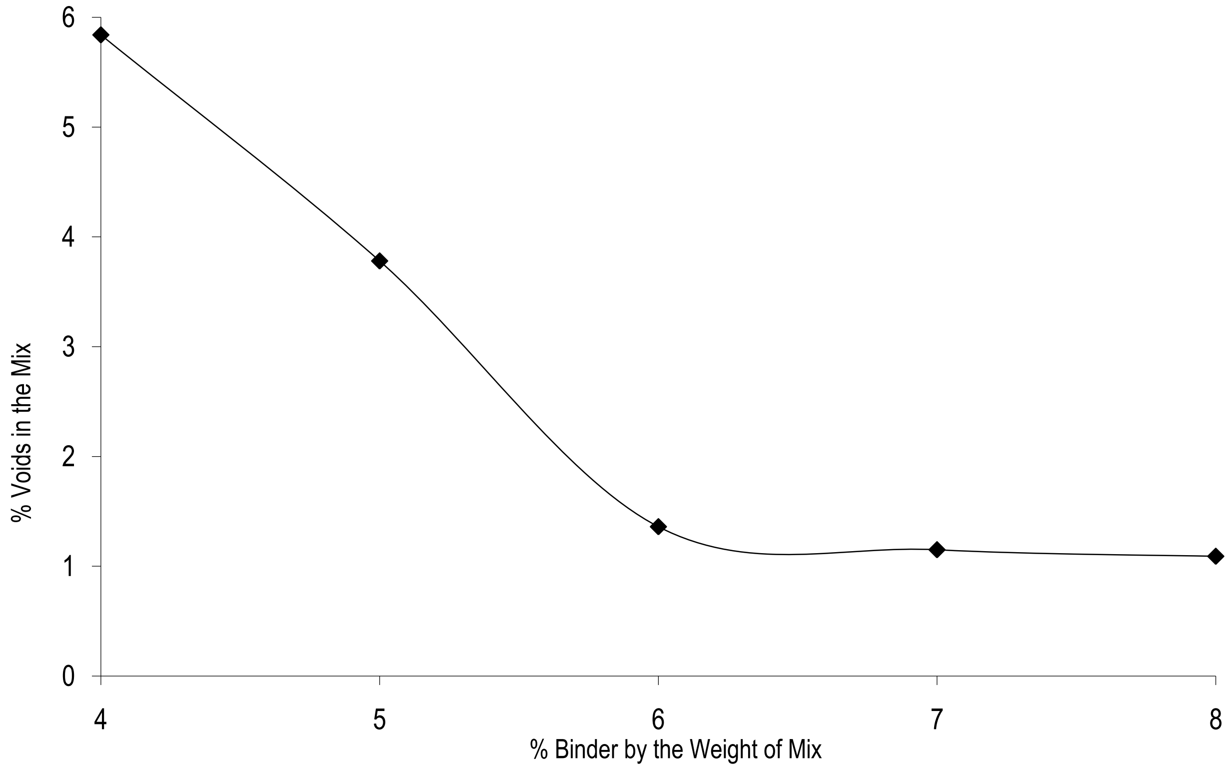

Fig 1.4.3 ; Voids in the Mix Vs % Bitumen Content

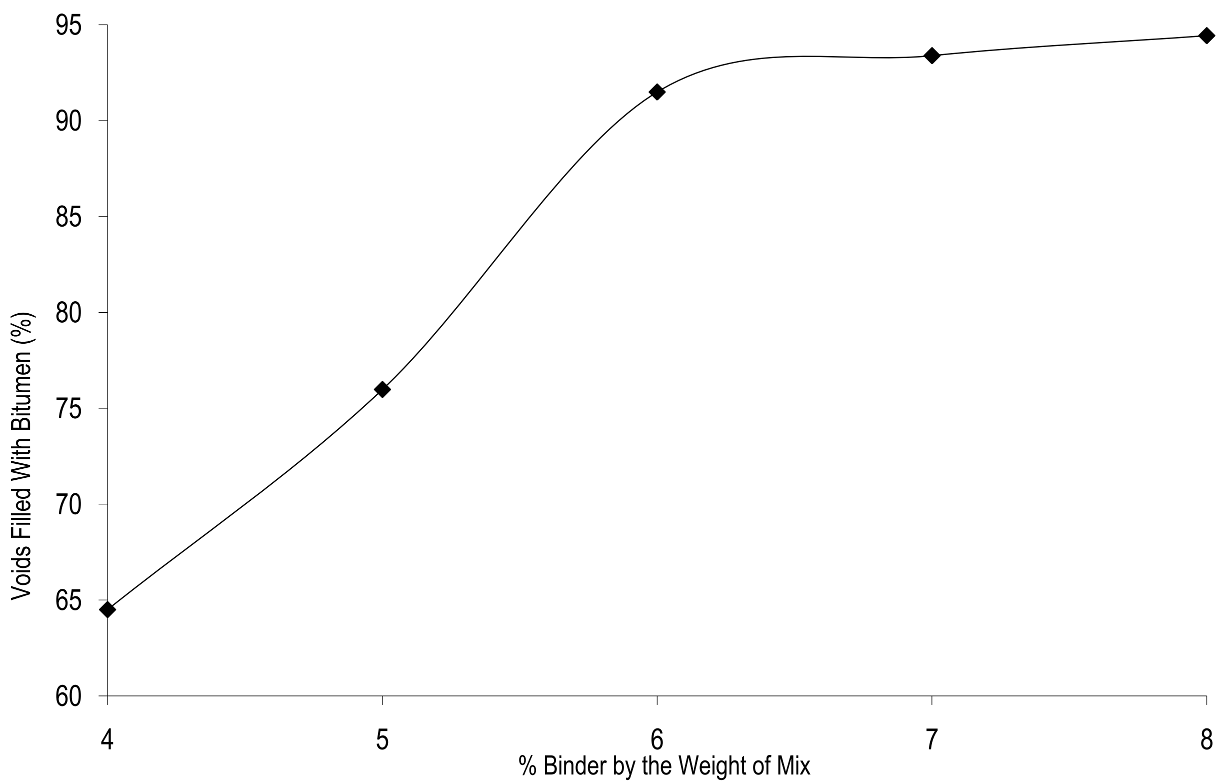

Fig 1.4.4 Voids Filled With Bitumen Vs % Bitumen Content

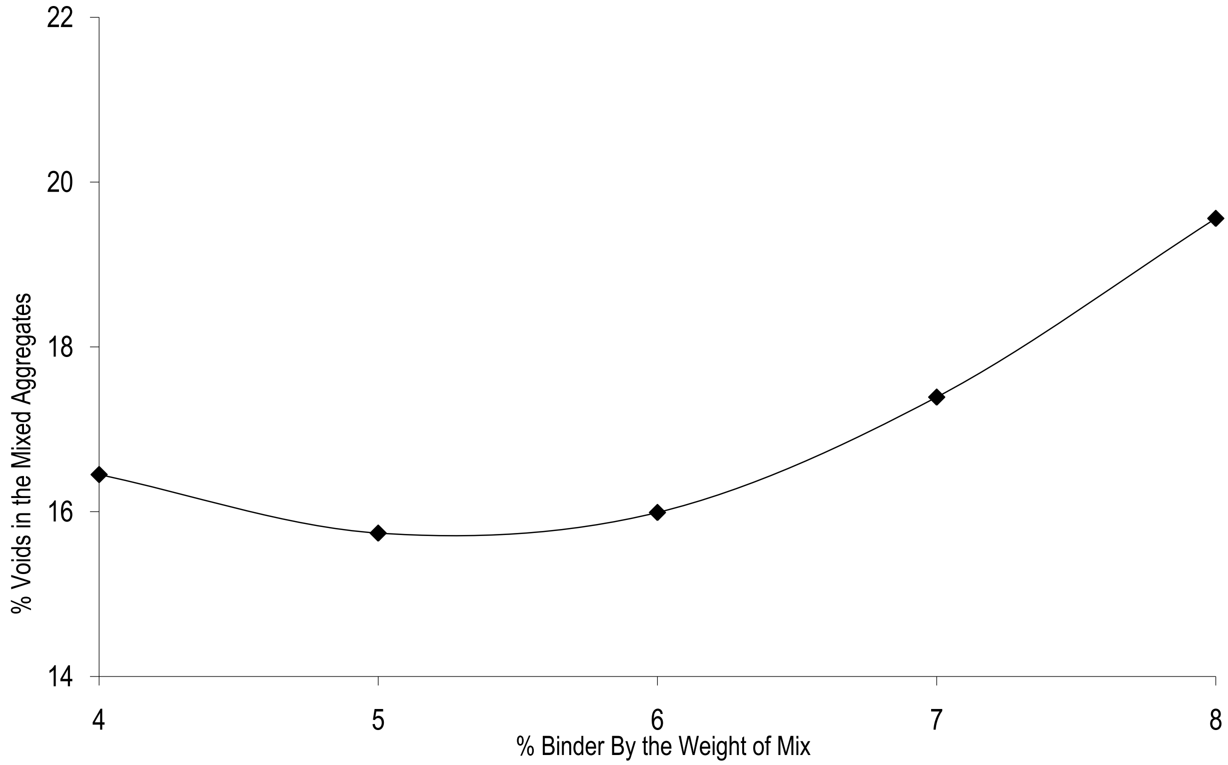

Fig 1.4.5 ; Voids in Mixed Aggregates Vs % Bitumen Content

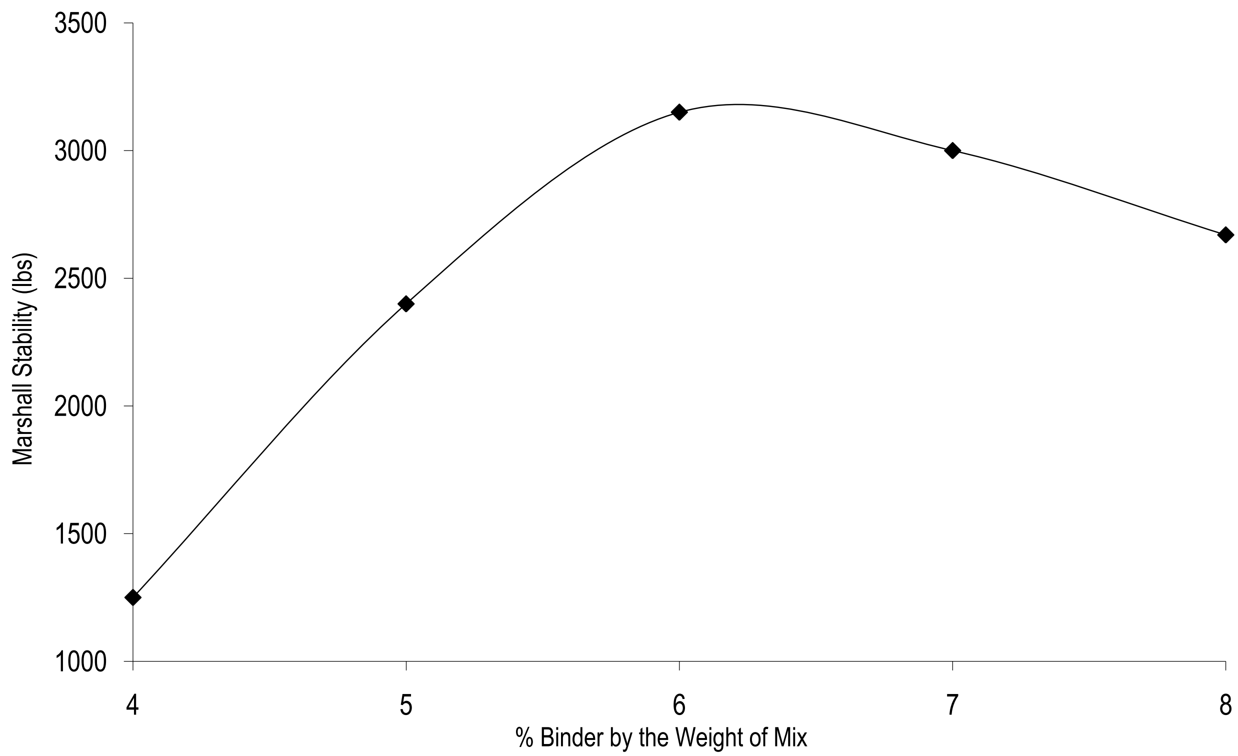

Fig 1.4.6 ; Marshall Stability Vs % Bitumen Content

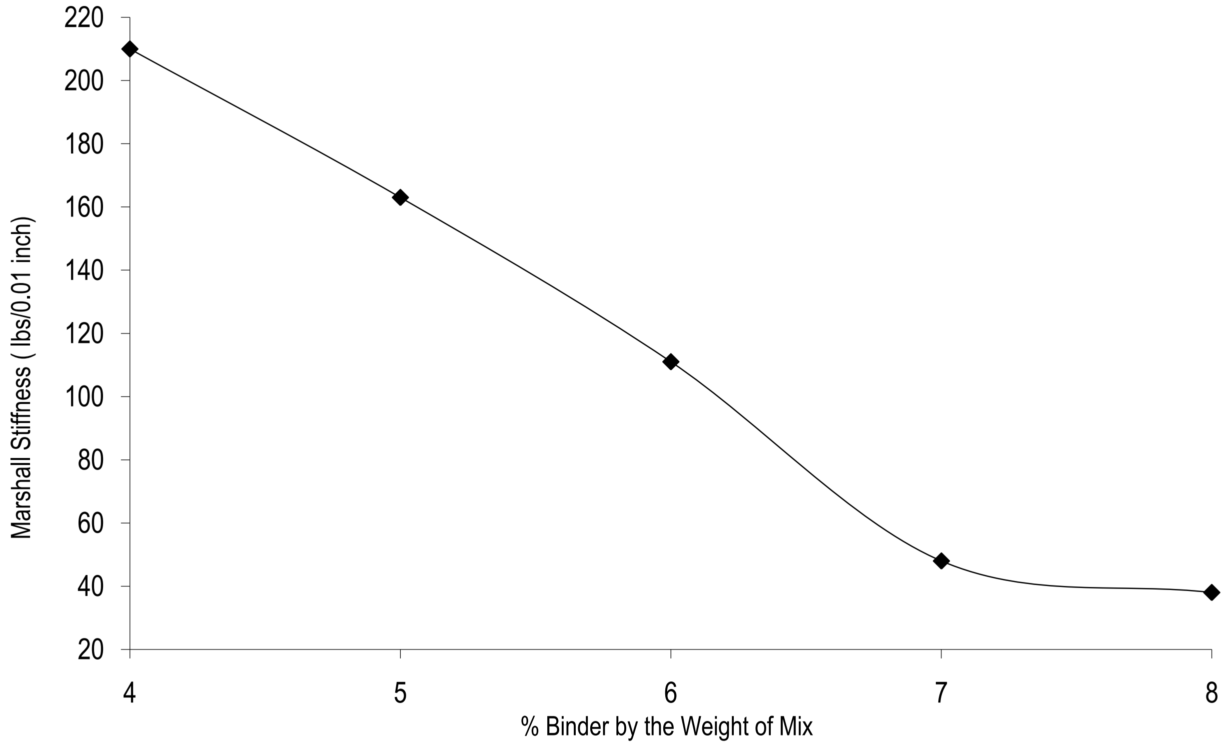

Fig 1.4.7 ; Marshall Stiffness Vs % Bitumen Content

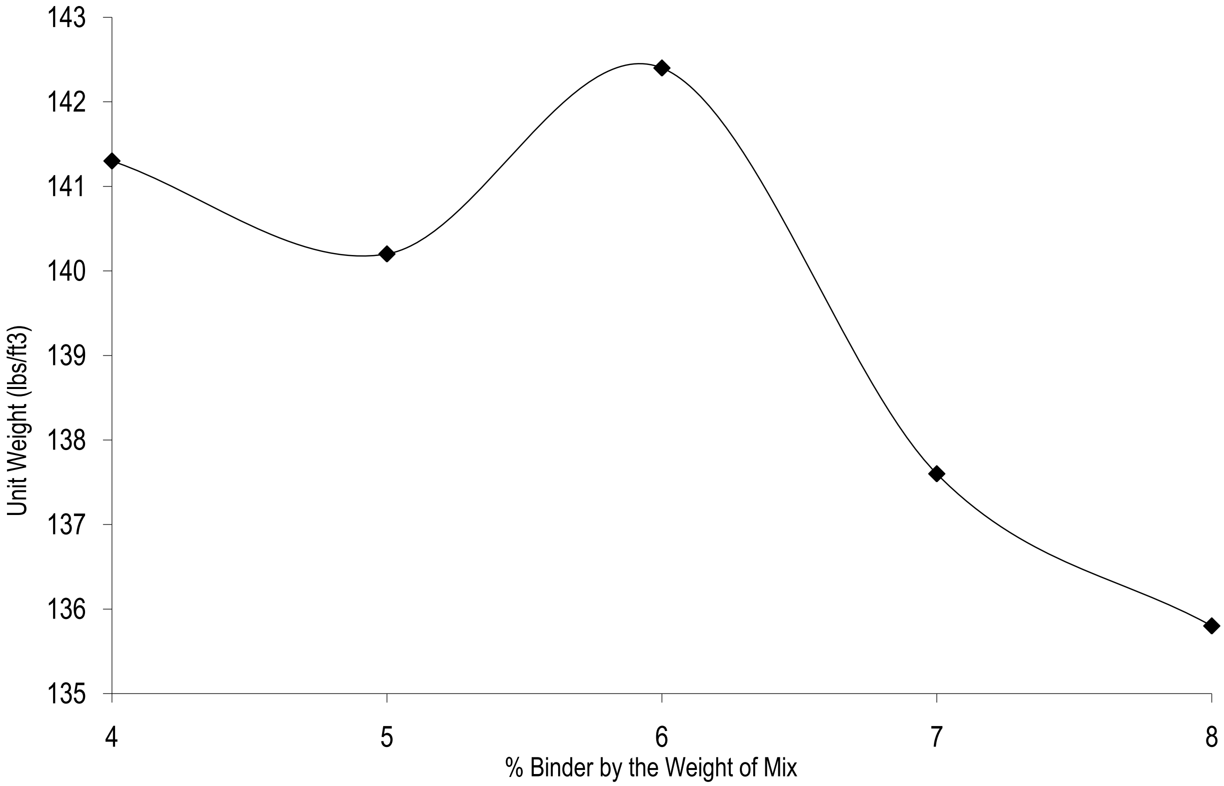

Fig 1.5.1 ; Unit Weight Vs % Bitumen Content

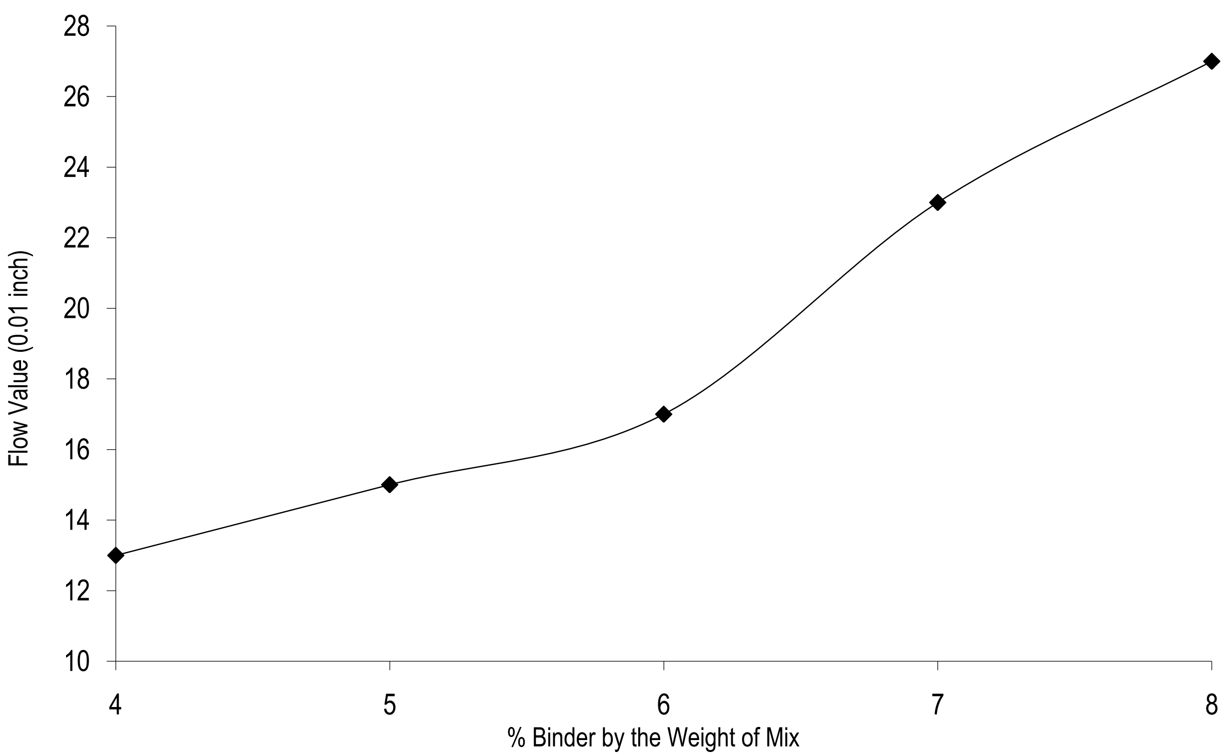

Fig 1.5.2 ; Flow Value Vs % Bitumen Content

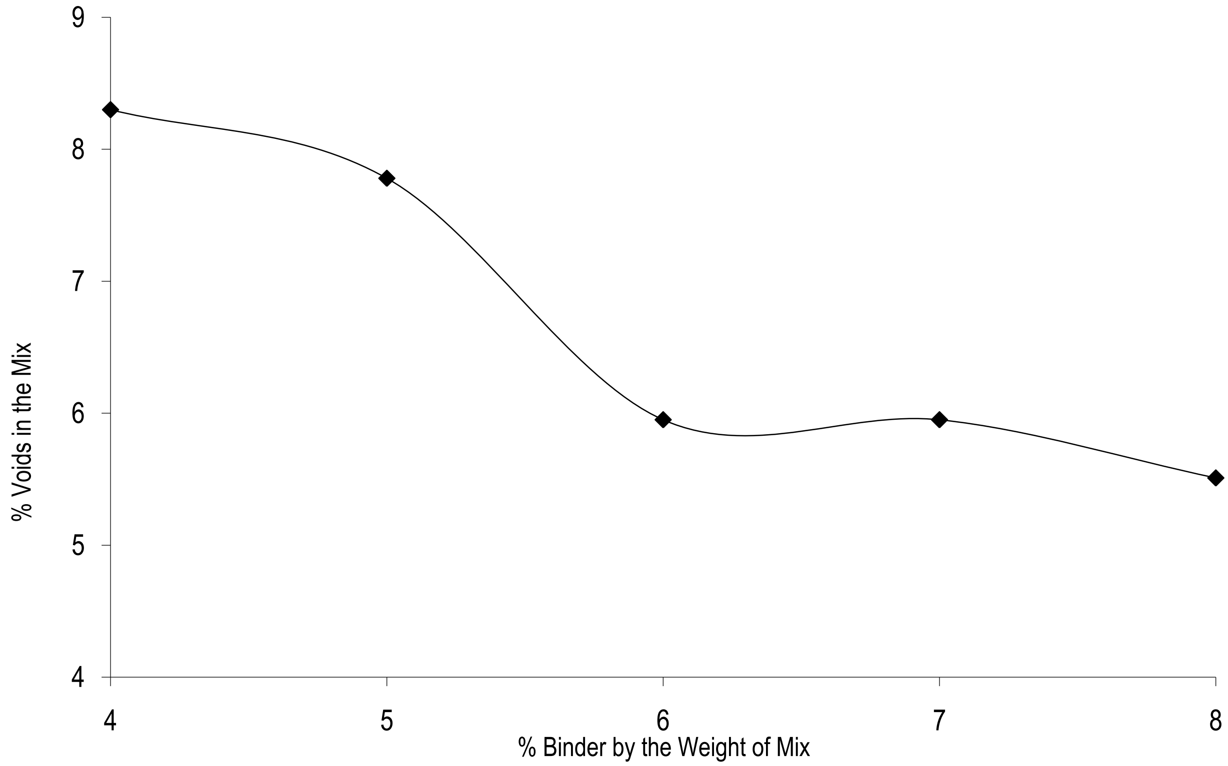

Fig 1.5.3 ; % Voids in the Mix Vs % Bitumen Content

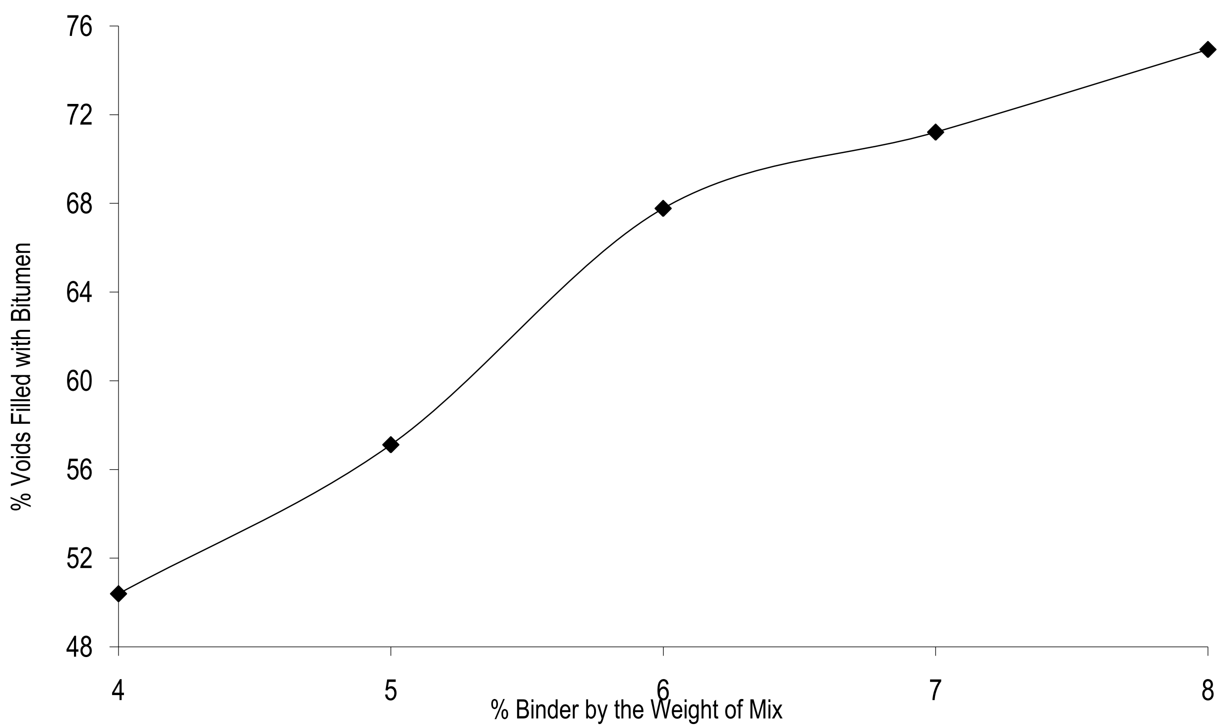

Fig 1.5.4 ; % Voids Filled With Bitumen Vs % Bitumen Content

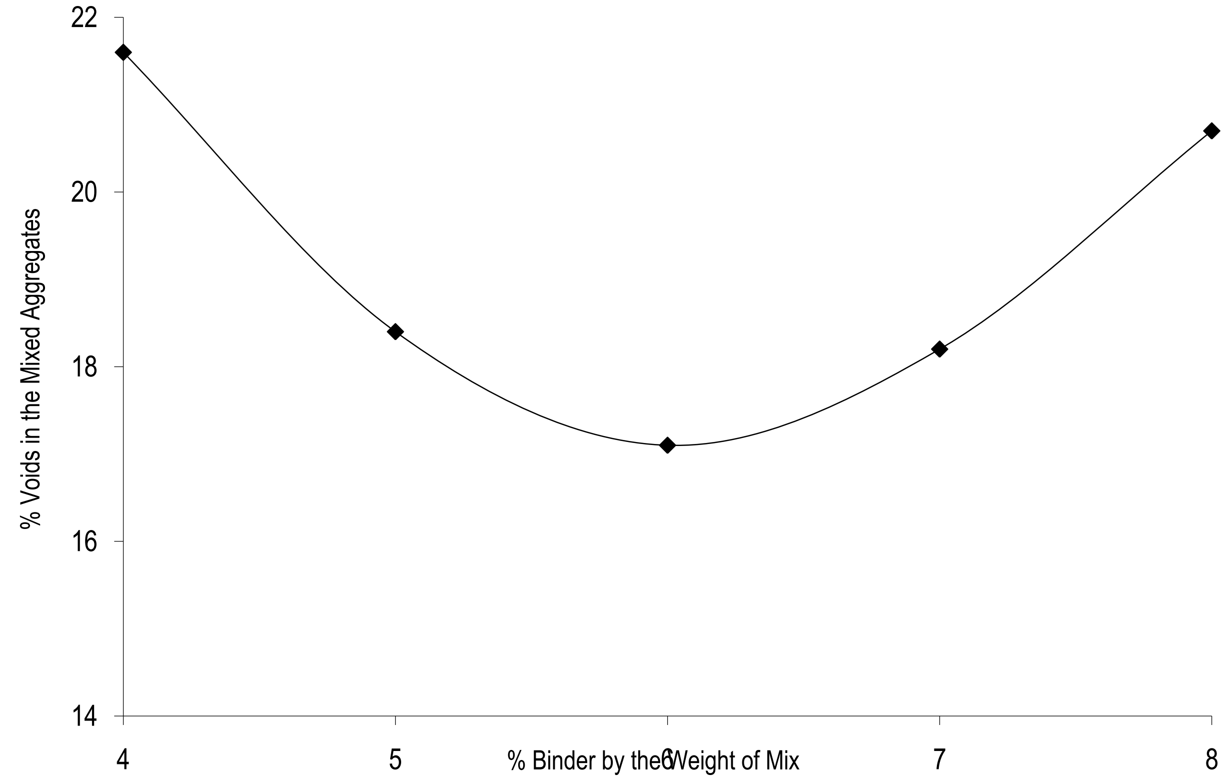

Fig 1.5.5 ; % Voids in Mixed Aggregates Vs % Bitumen Content

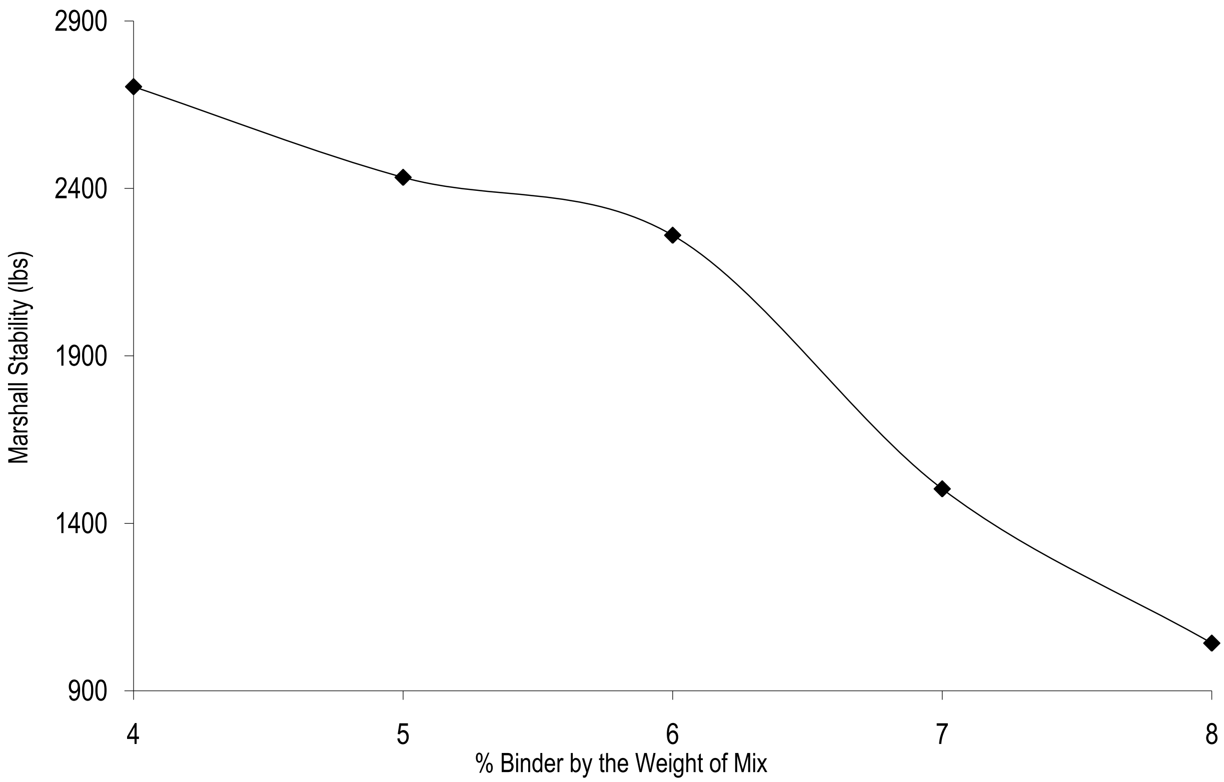

Fig 1.5.6 ; Marshall Stability Vs % Bitumen Content

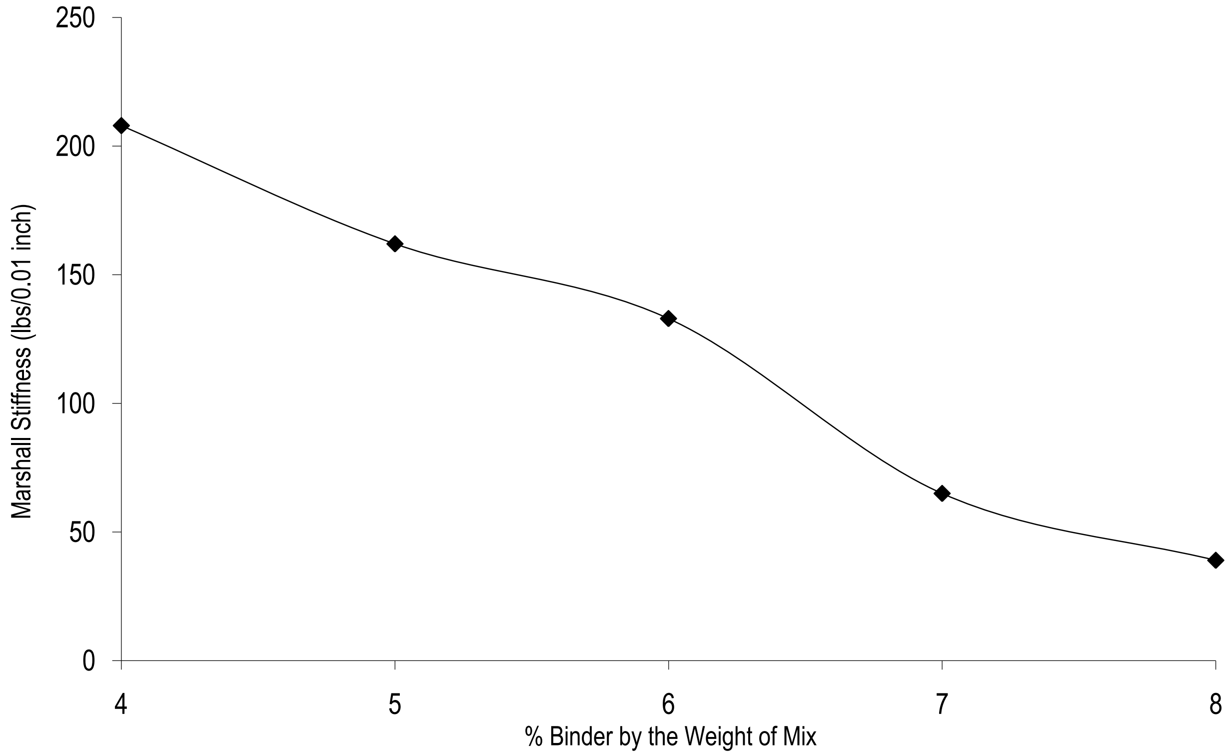

Fig 1.5.7 ; Marshall Stiffness Vs % Bitumen Content

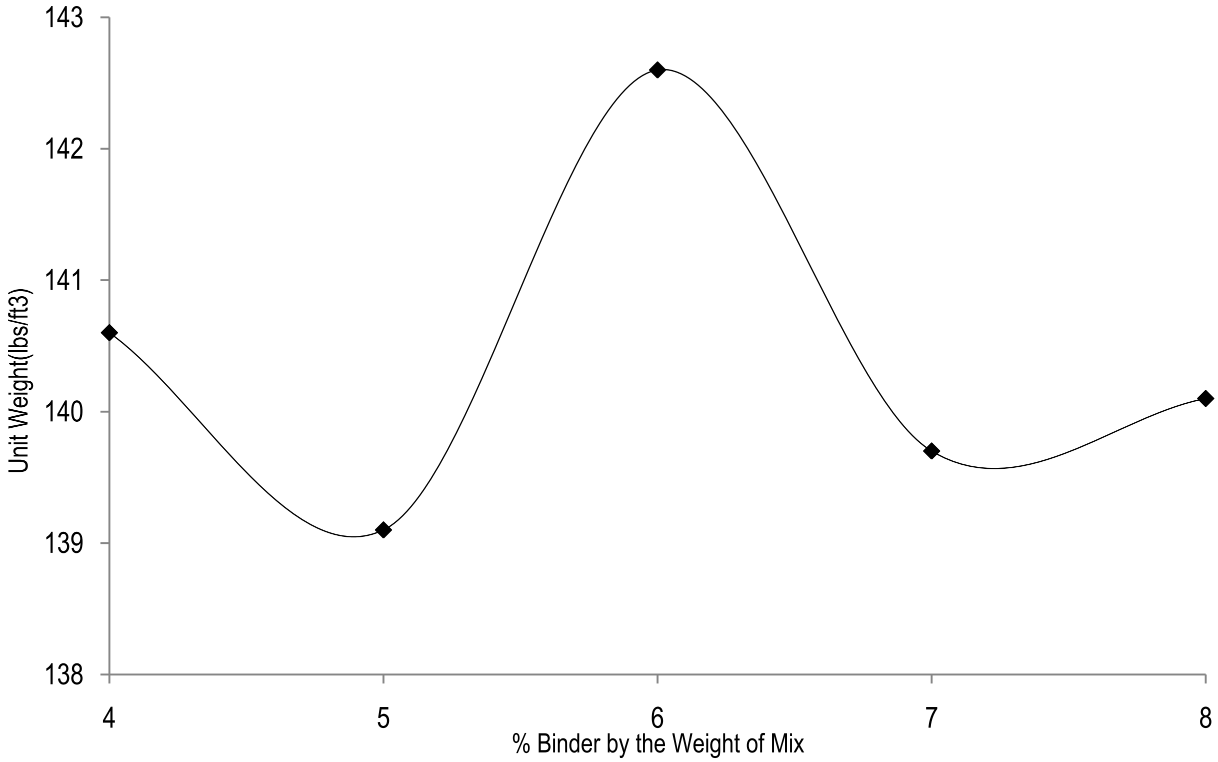

Fig 1.6.1 ; Unit Weight Vs % Bitumen Content

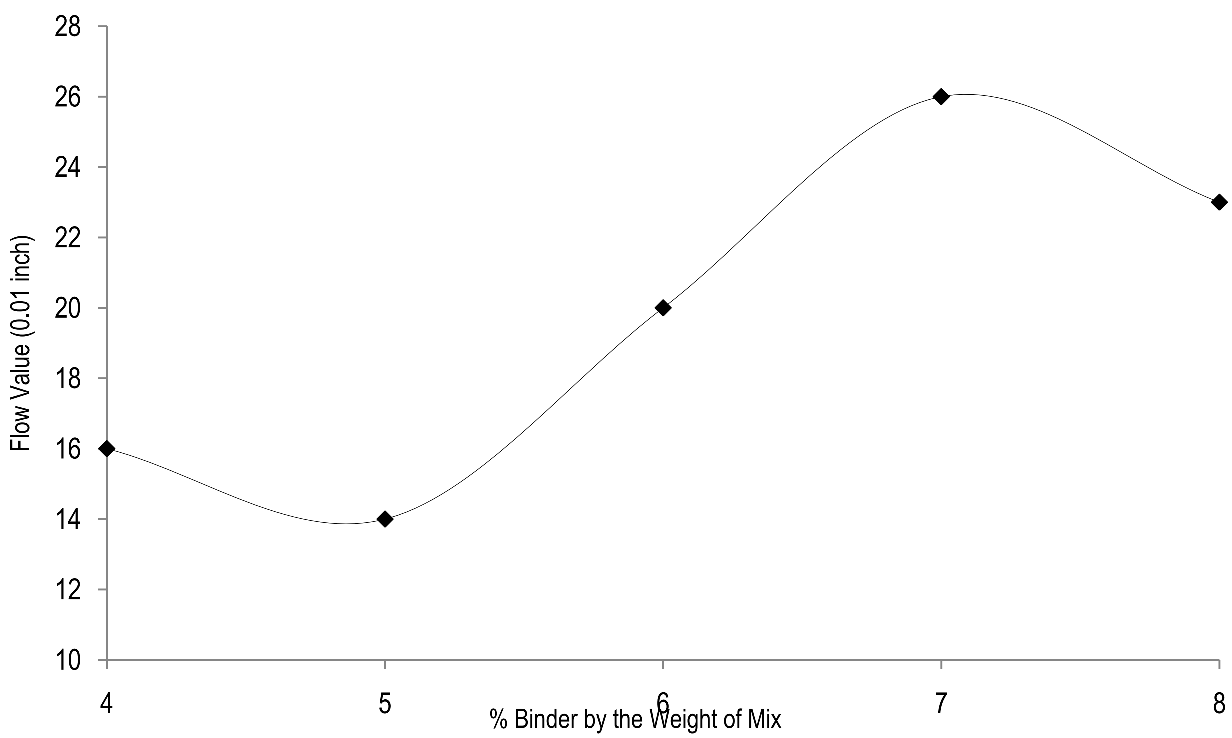

Fig 1.6.2 ; Flow Value Vs % Bitumen Content

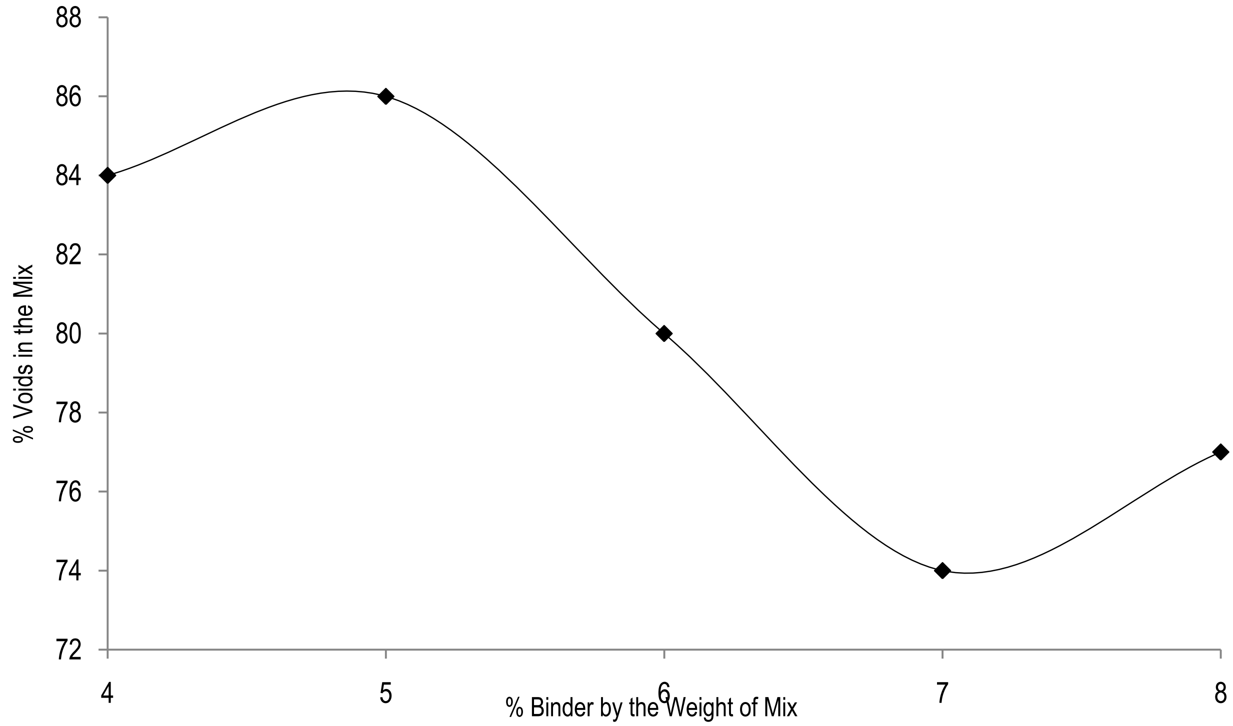

Fig 1.6.3 ; % Voids in the Mix Vs % Bitumen Content

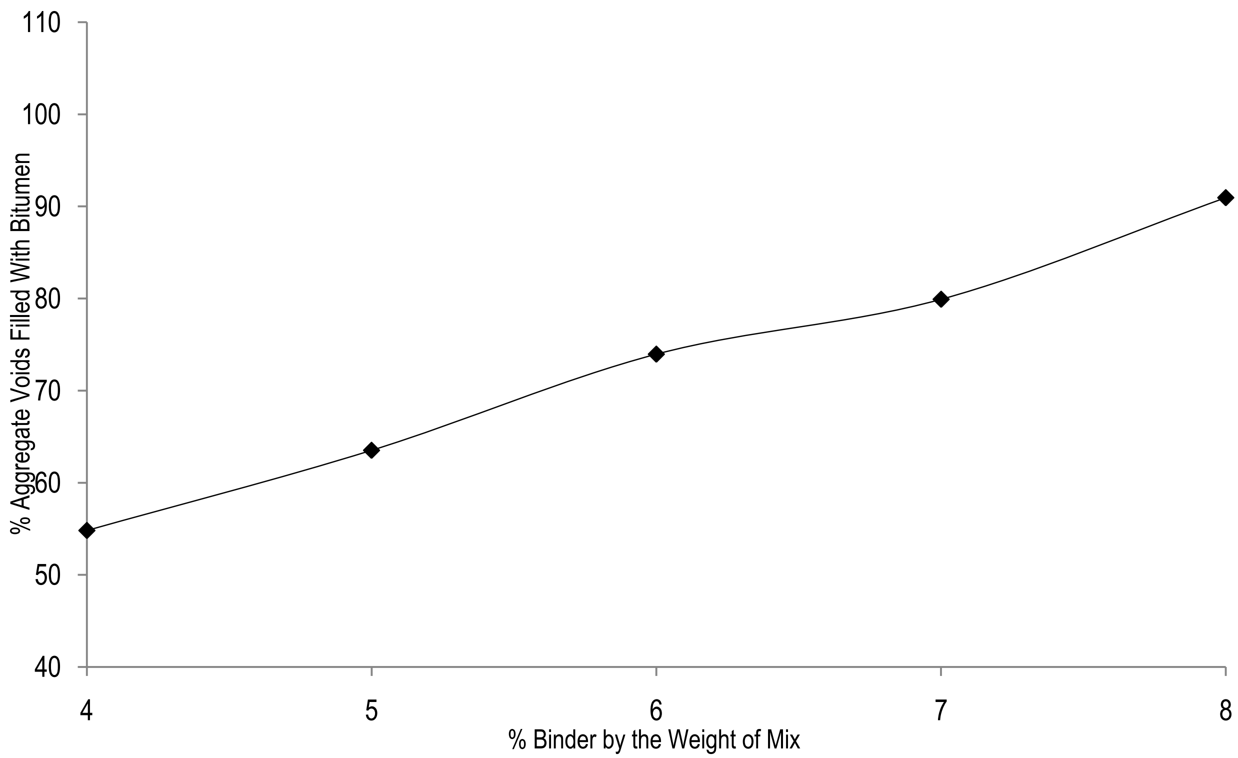

Fig 1.6.4 ; % Aggregate Voids Filled With Bitumen Vs % Bitumen Content

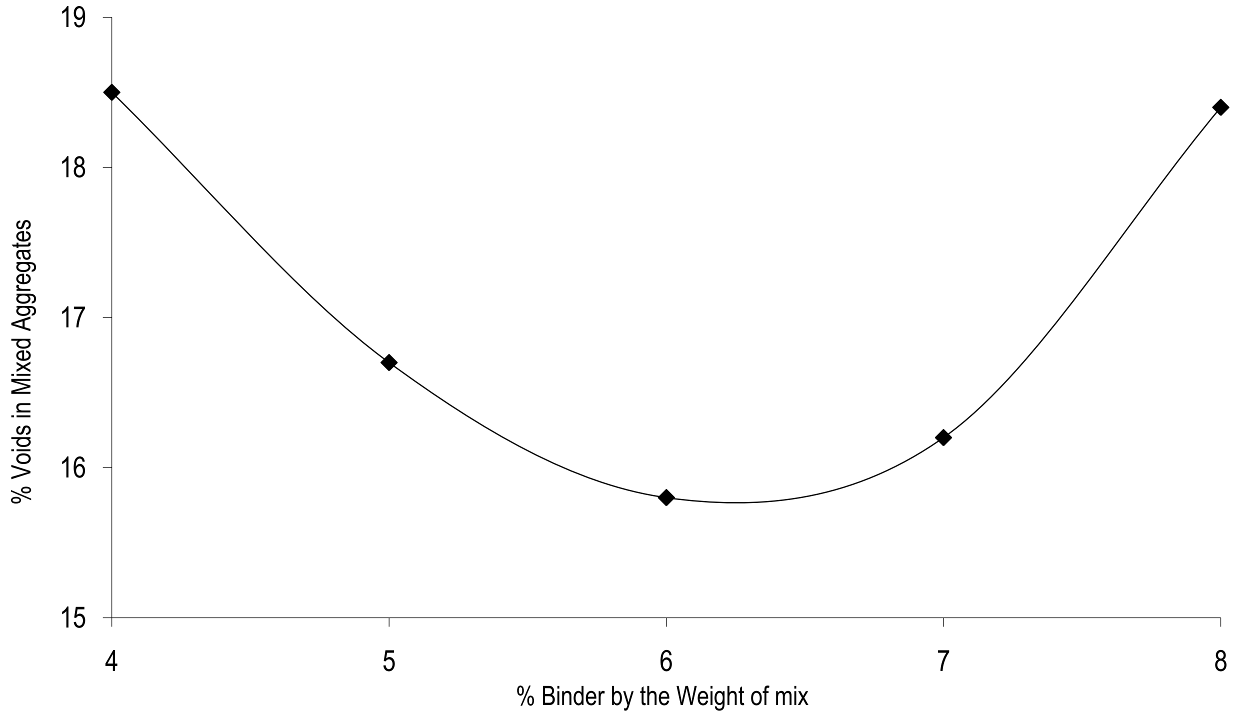

Fig 1.6.5 ; % Voids in Mixed Aggregates Vs Bitumen Content

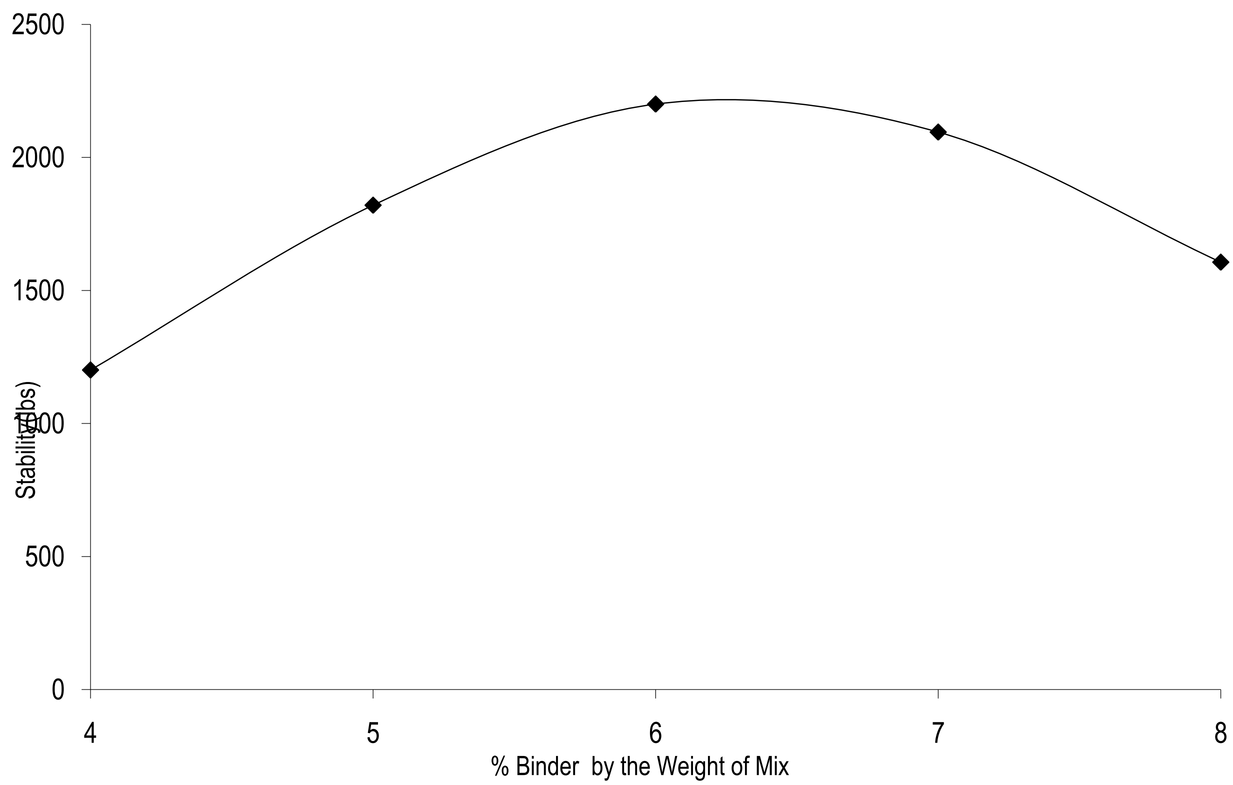

Fig 1.6.6 ; Stability vs % Bitumen Content

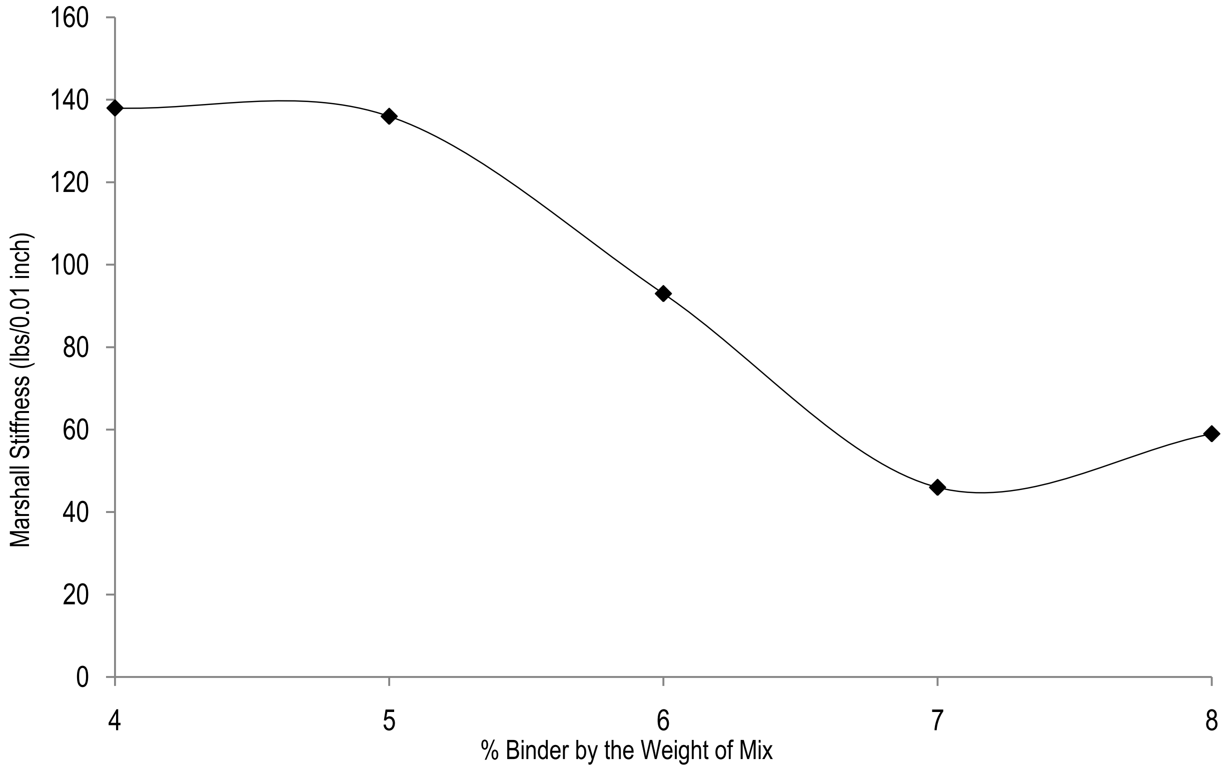

Fig 1.6.7 ; Marshall Stiffness Vs % Bitumen Content

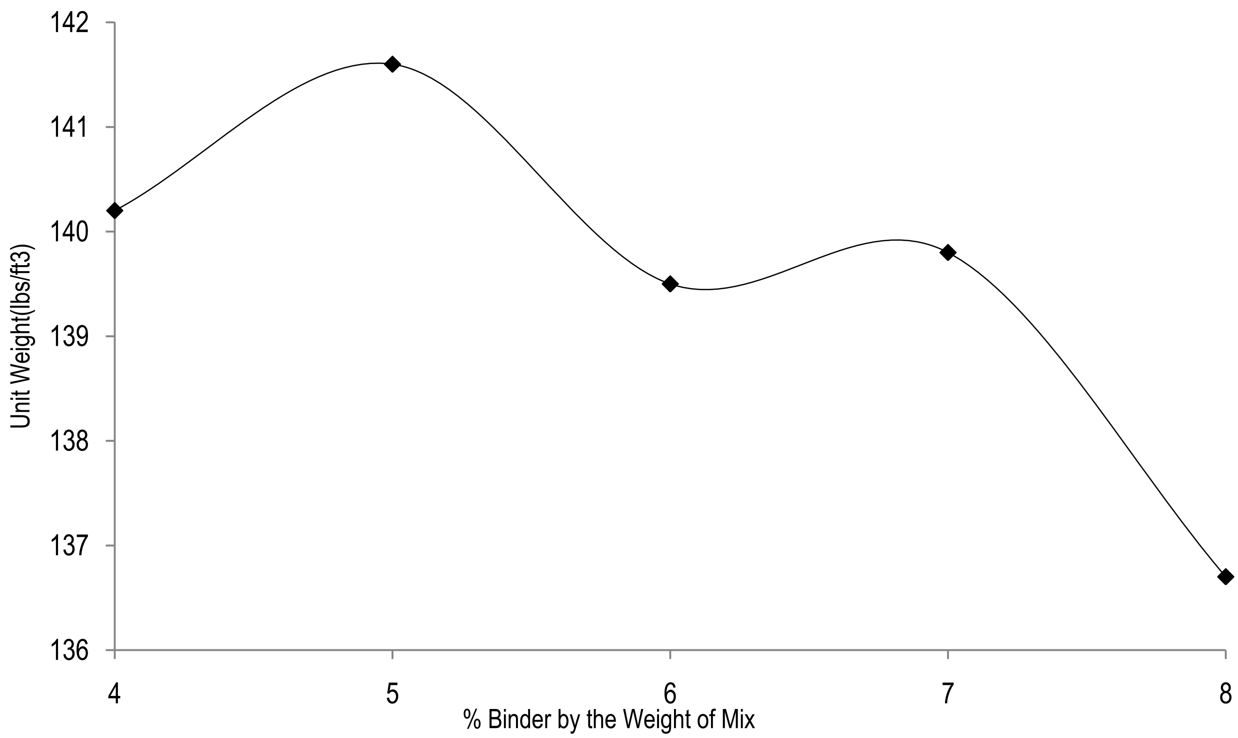

Fig 1.7.1 ; Unit Weight Vs % Bitumen Content

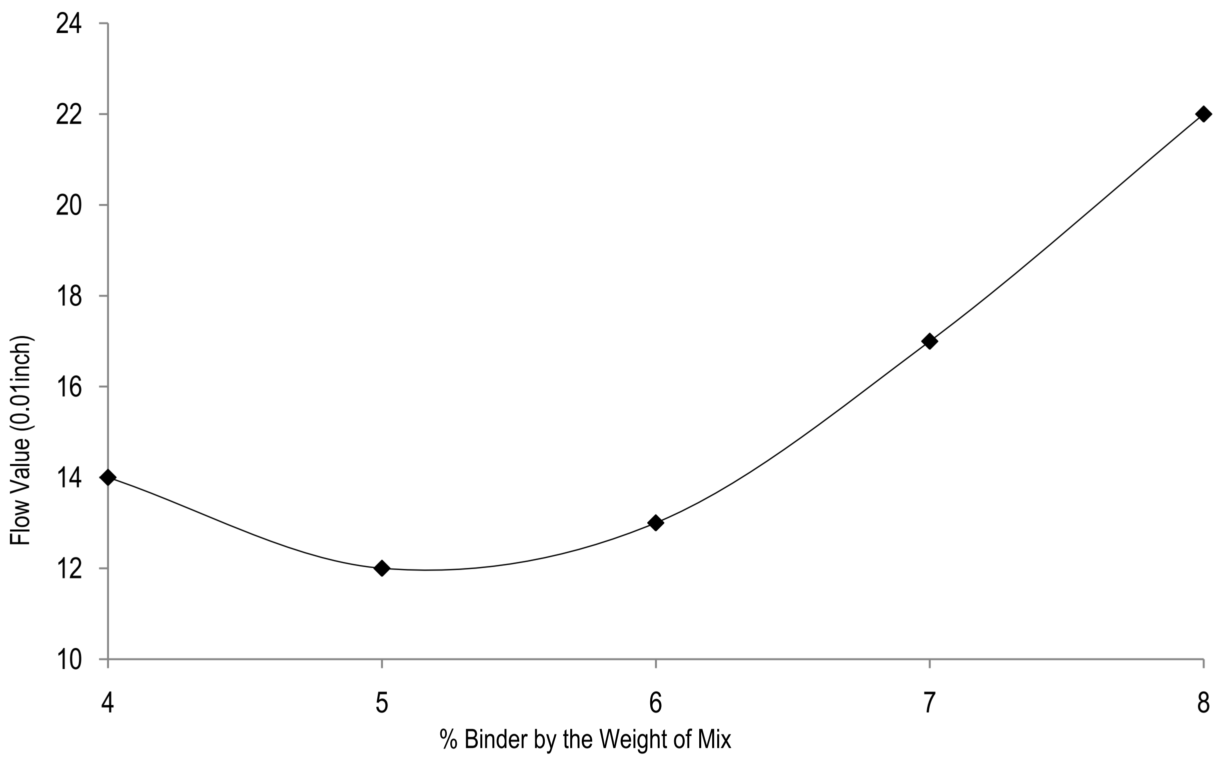

Fig 1.7.2 ; Flow Value Vs % Bitumen Content

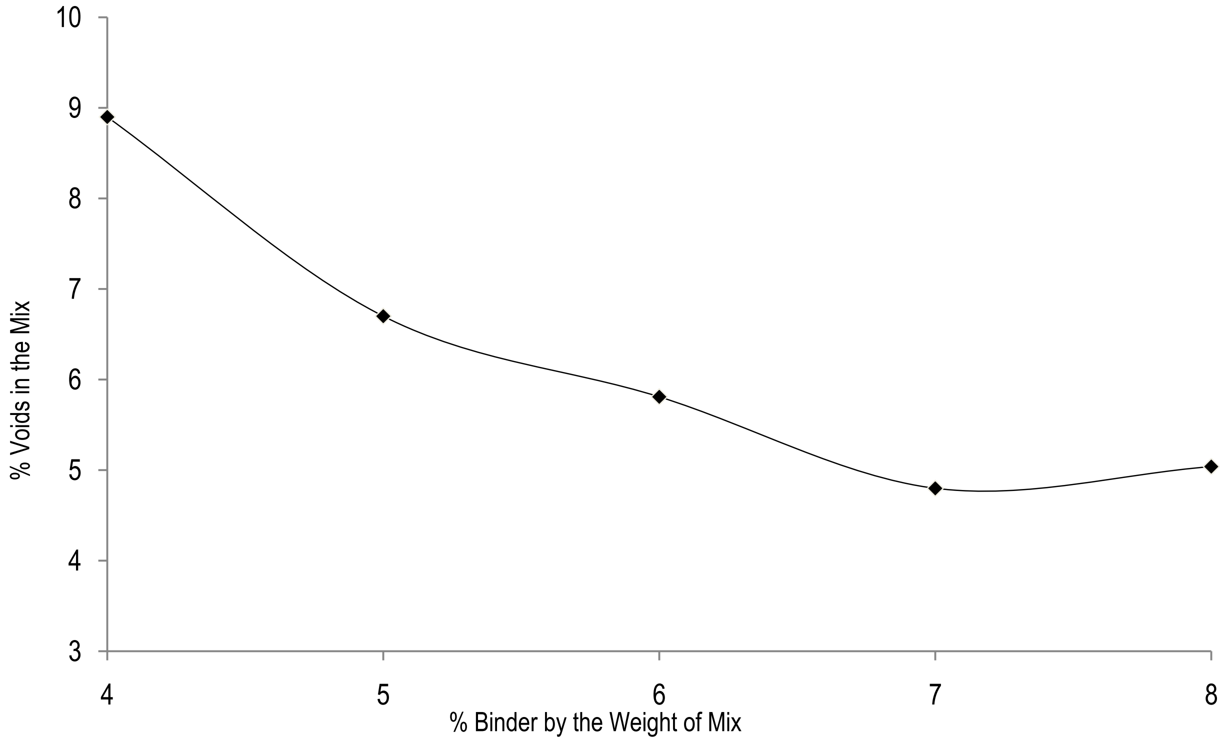

Fig 1.7.3 ; % Voids in the Mix Vs % Bitumen Content

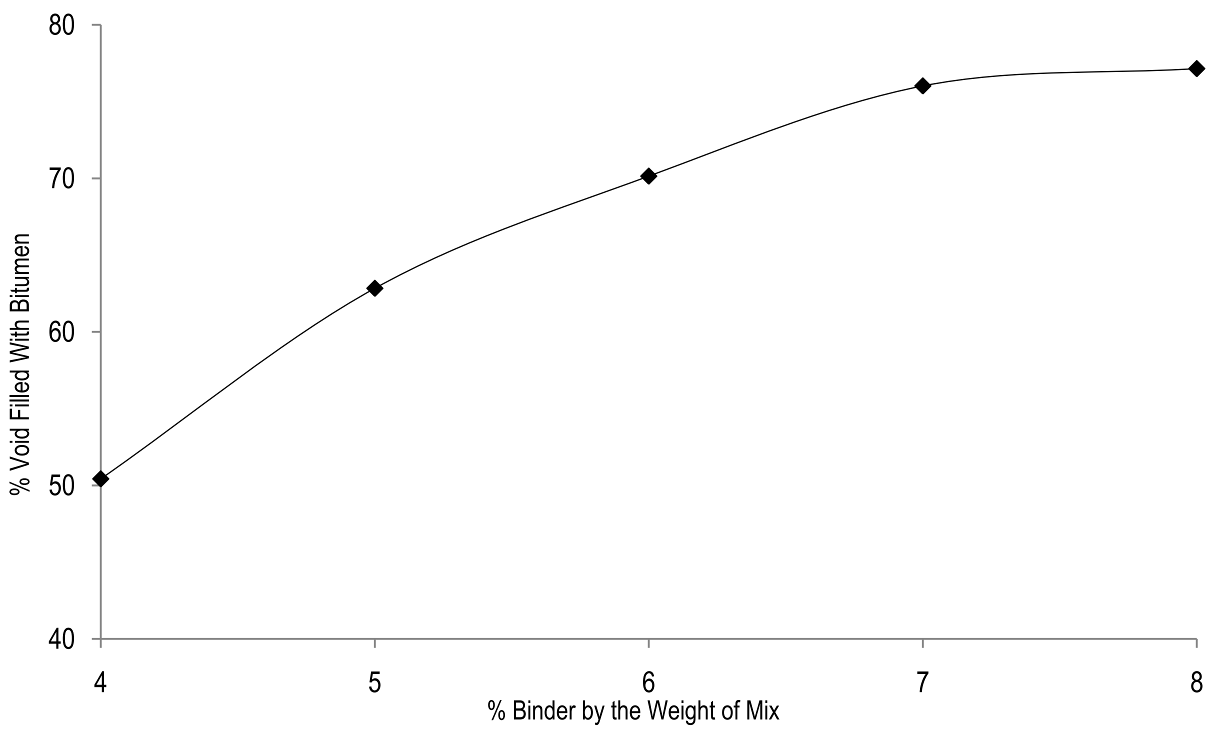

Fig 1.7.4 ; % Void Filled With Bitumen Vs % Bitumen Content

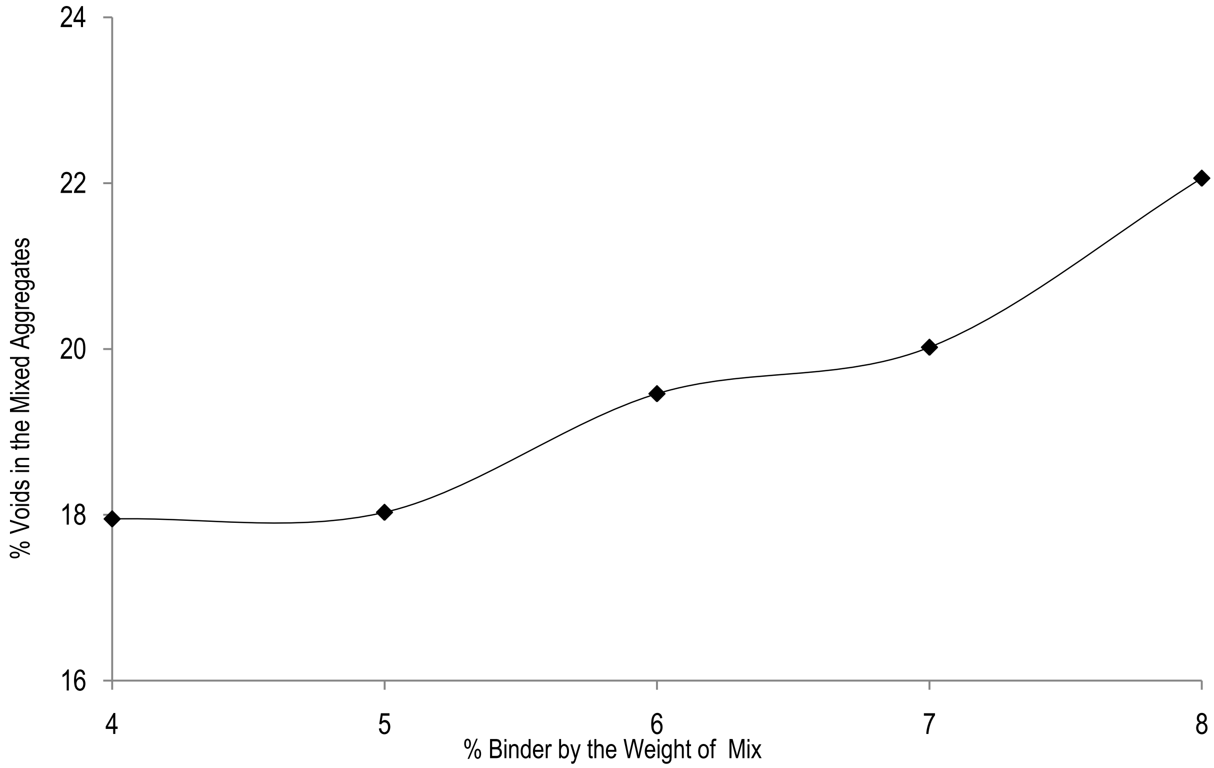

Fig 1.7.5 ; % Voids in the Mixed Aggregates

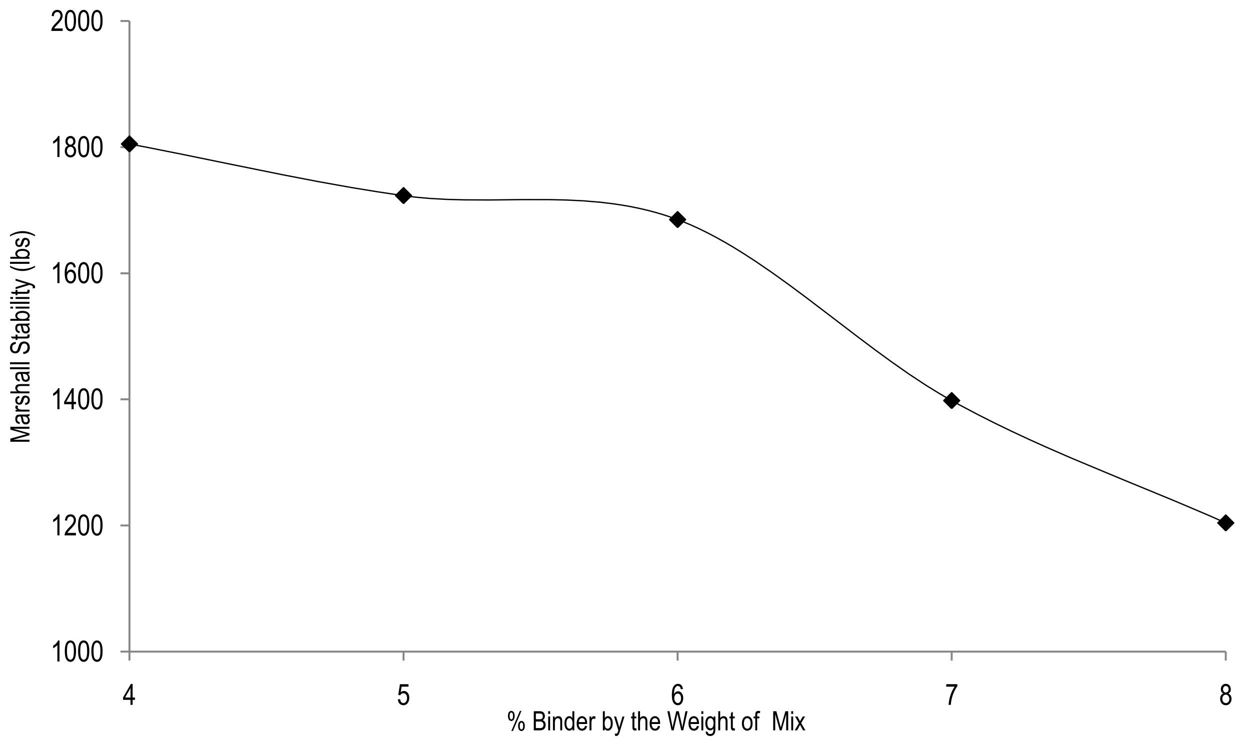

Fig 1.7.6 ; Marshall Stability Vs % Bitumen Content

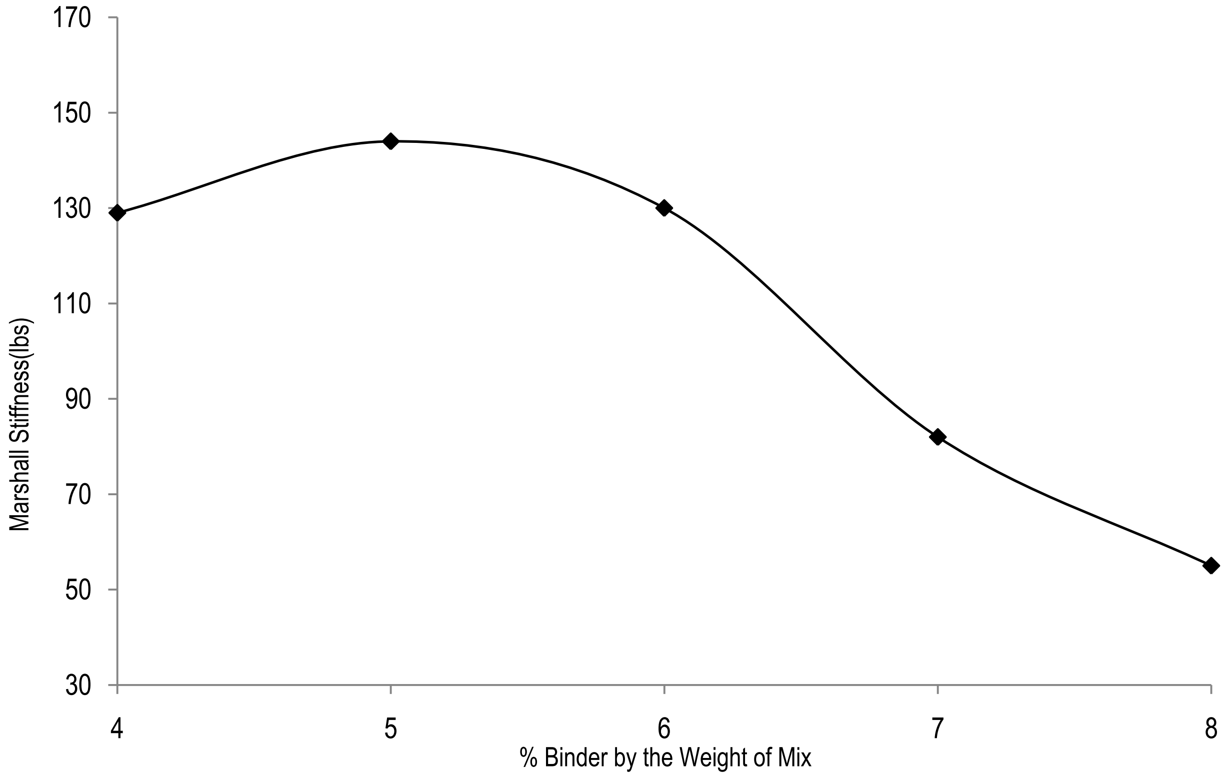

Fig 1.7.7 ; Marshall Stiffness Vs % Bitumen Content

]

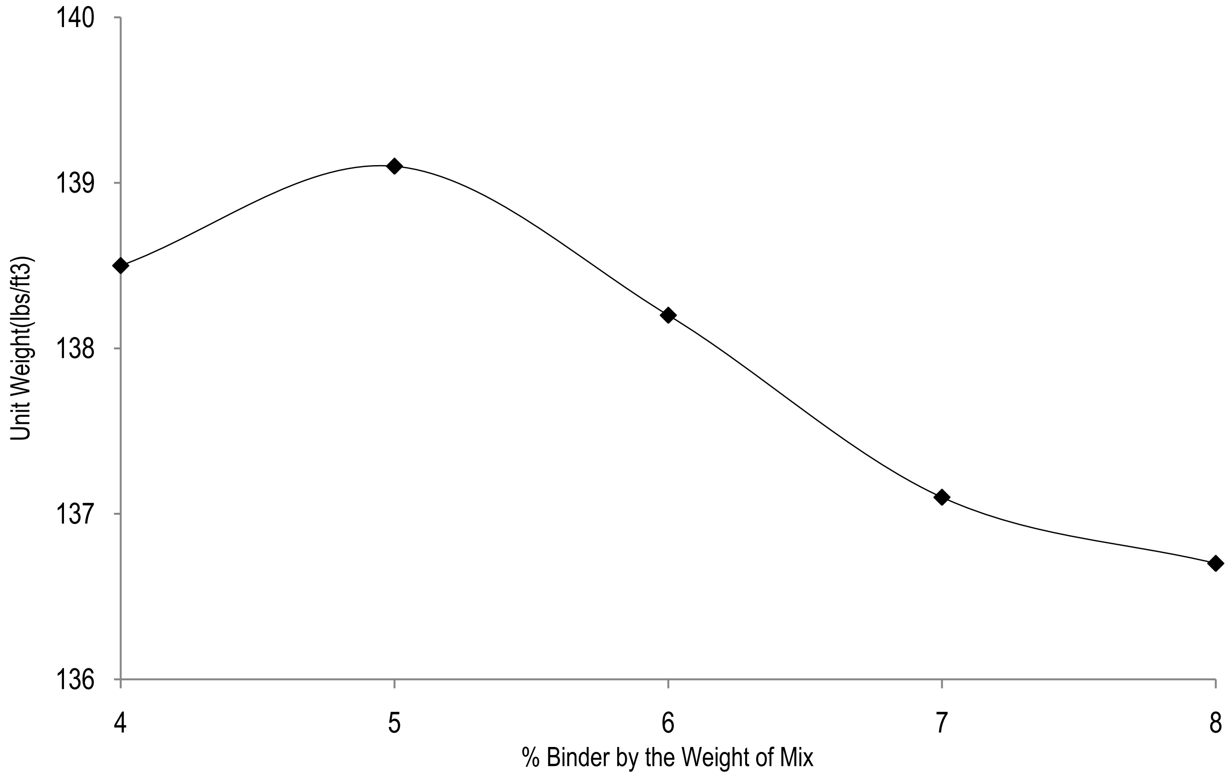

Fig 1.8.1;Unit Weight Vs % Bitumen Content

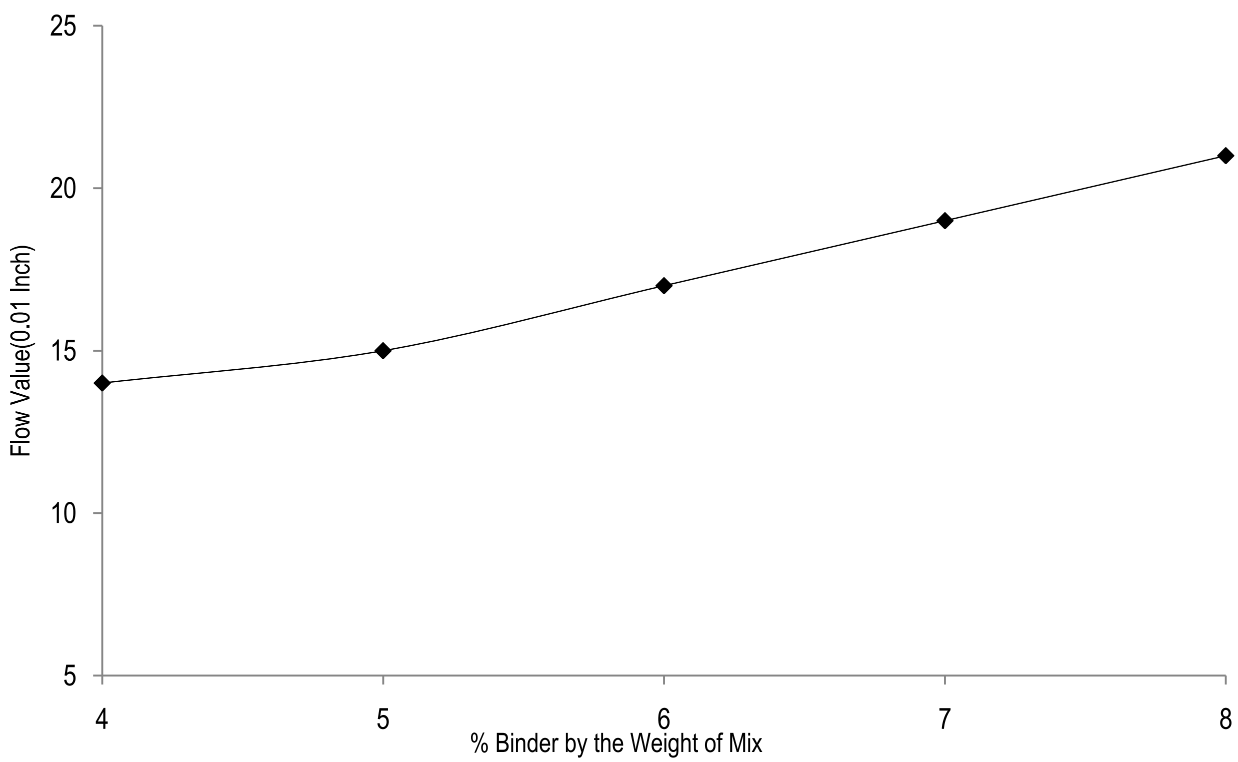

Fig 1.8.2 ; Flow Value Vs % Bitumen Content

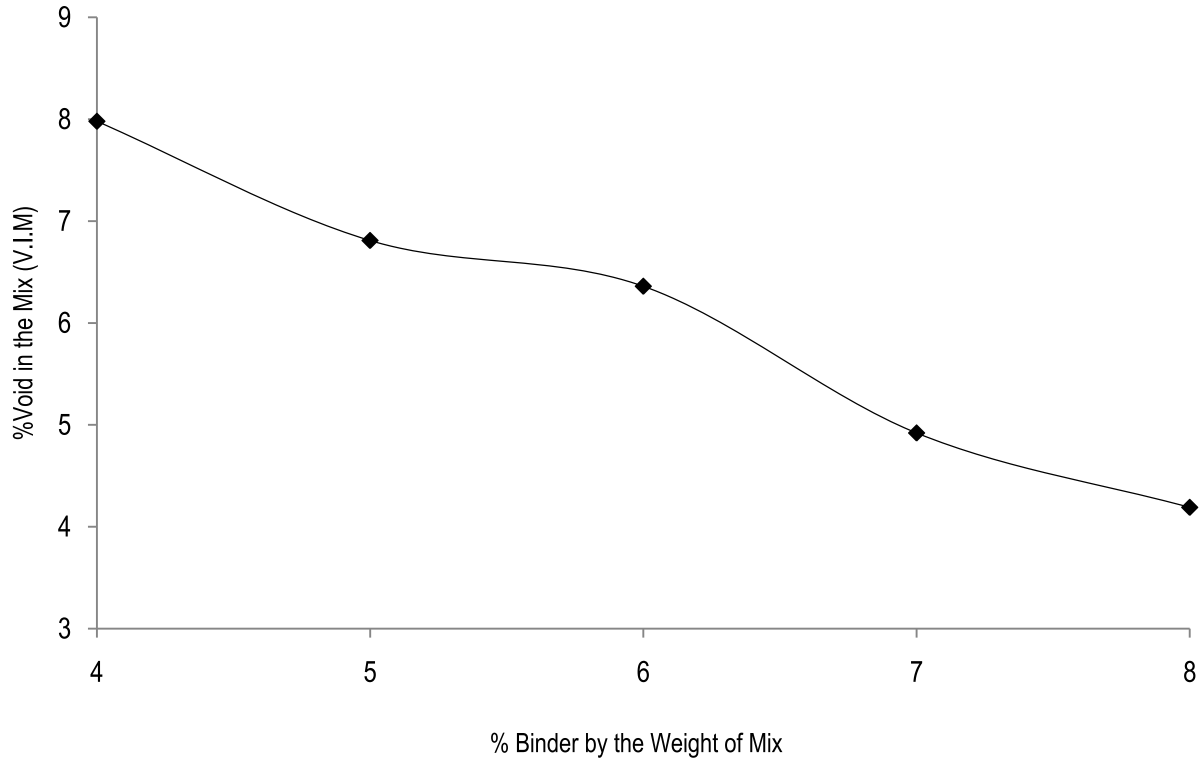

Fig 1.8.3 ; % Voids in the Mix Vs % Bitumen Content

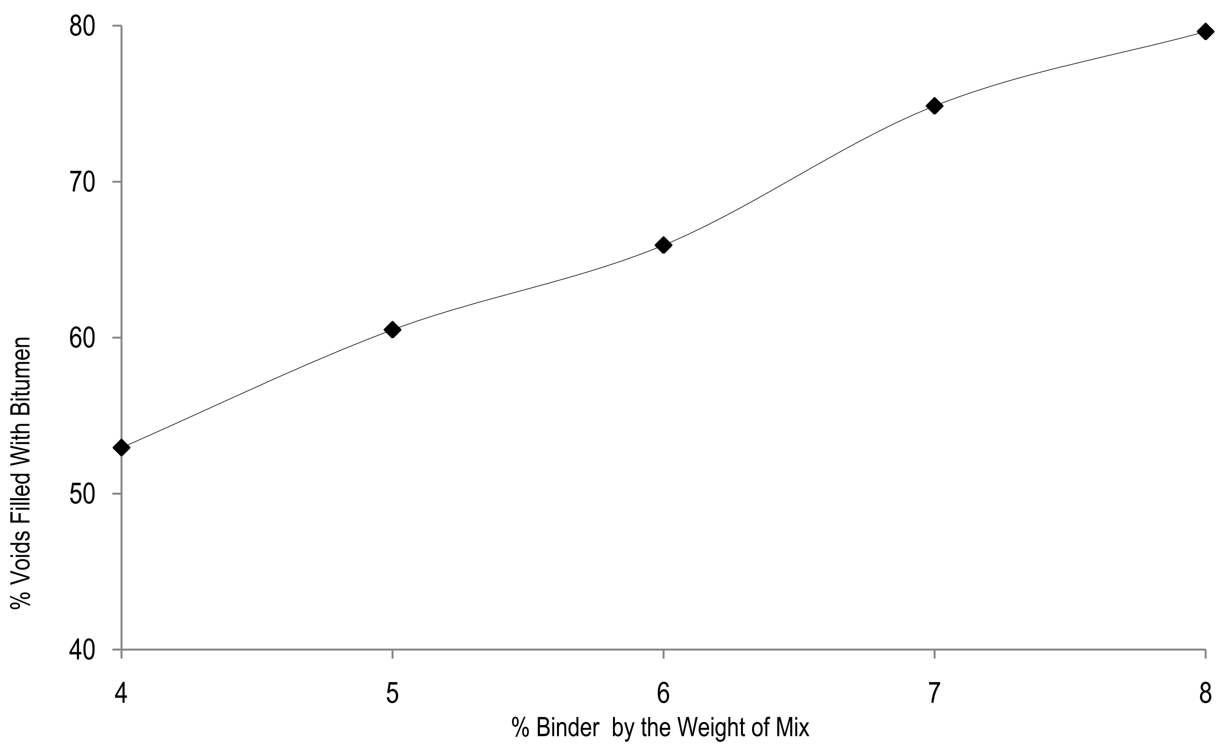

Fig 1.8.4 ; % Voids Filled With Bitumen Vs % Bitumen Content

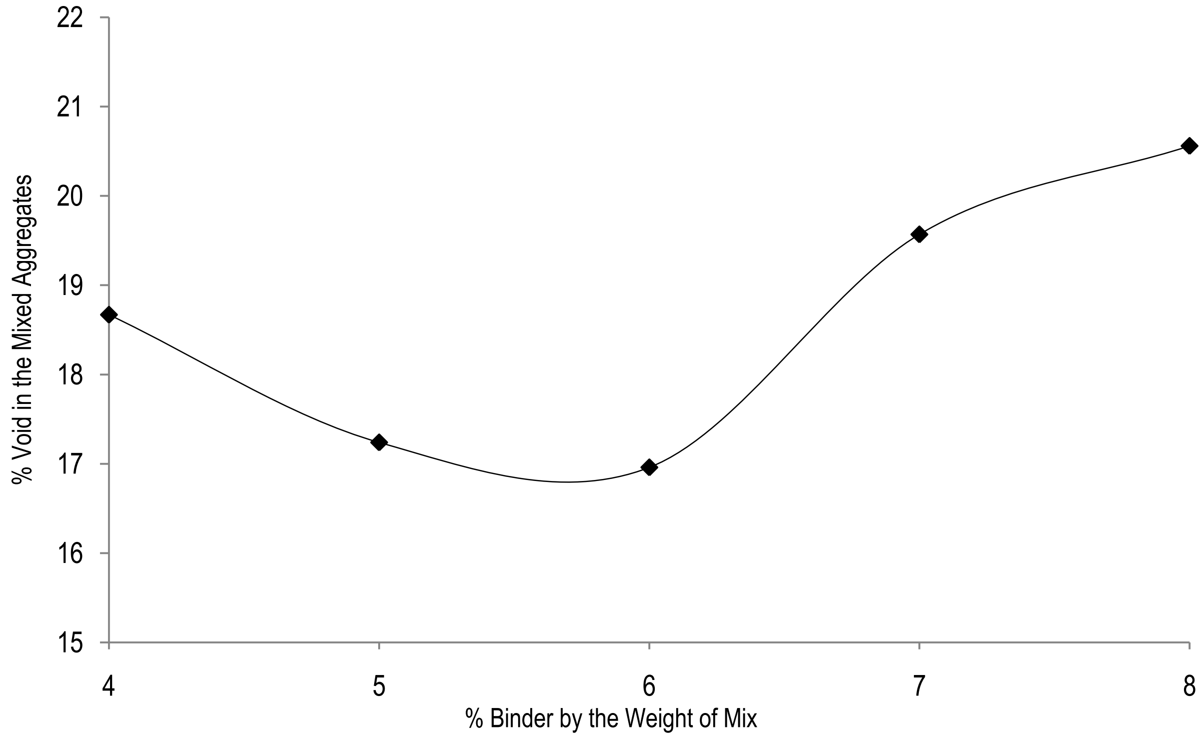

Fig 1.8.5 ; % Voids in the Mixed Aggregates Vs % Bitumen Content

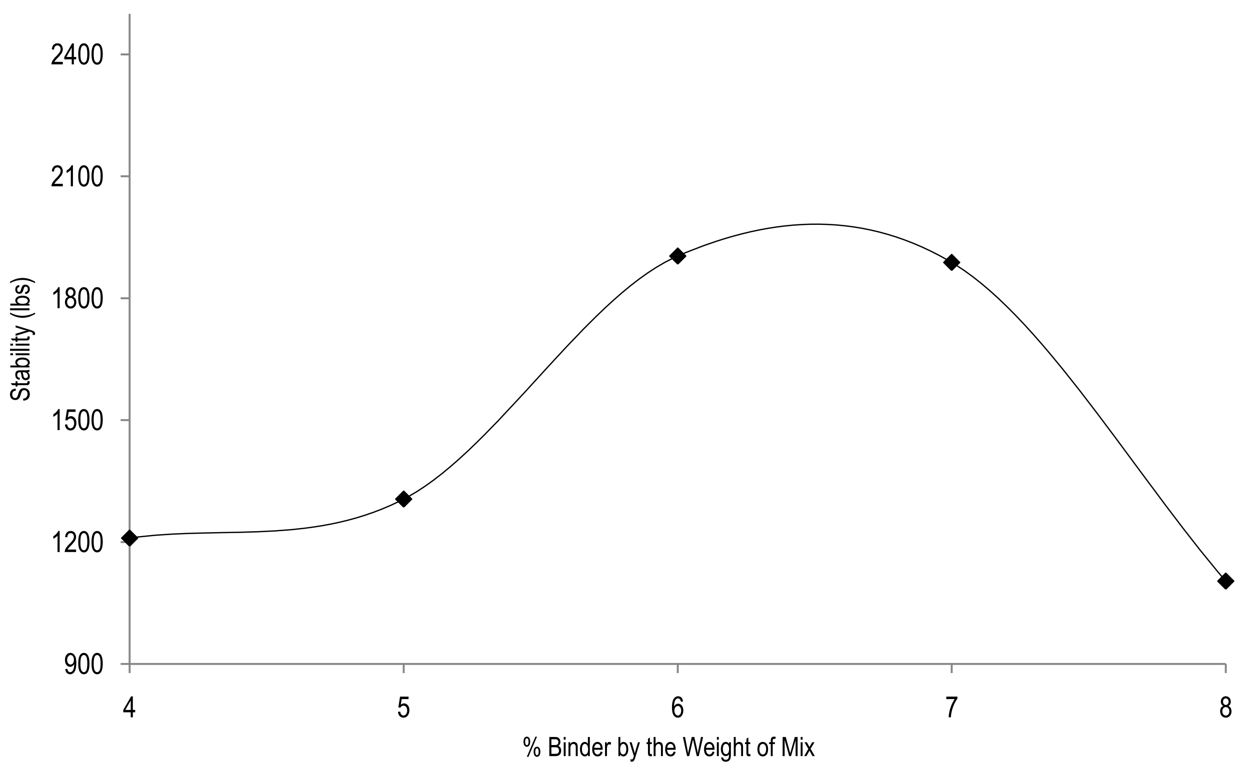

Fig 1.8.6 ; Stability Vs % Bitumen Content

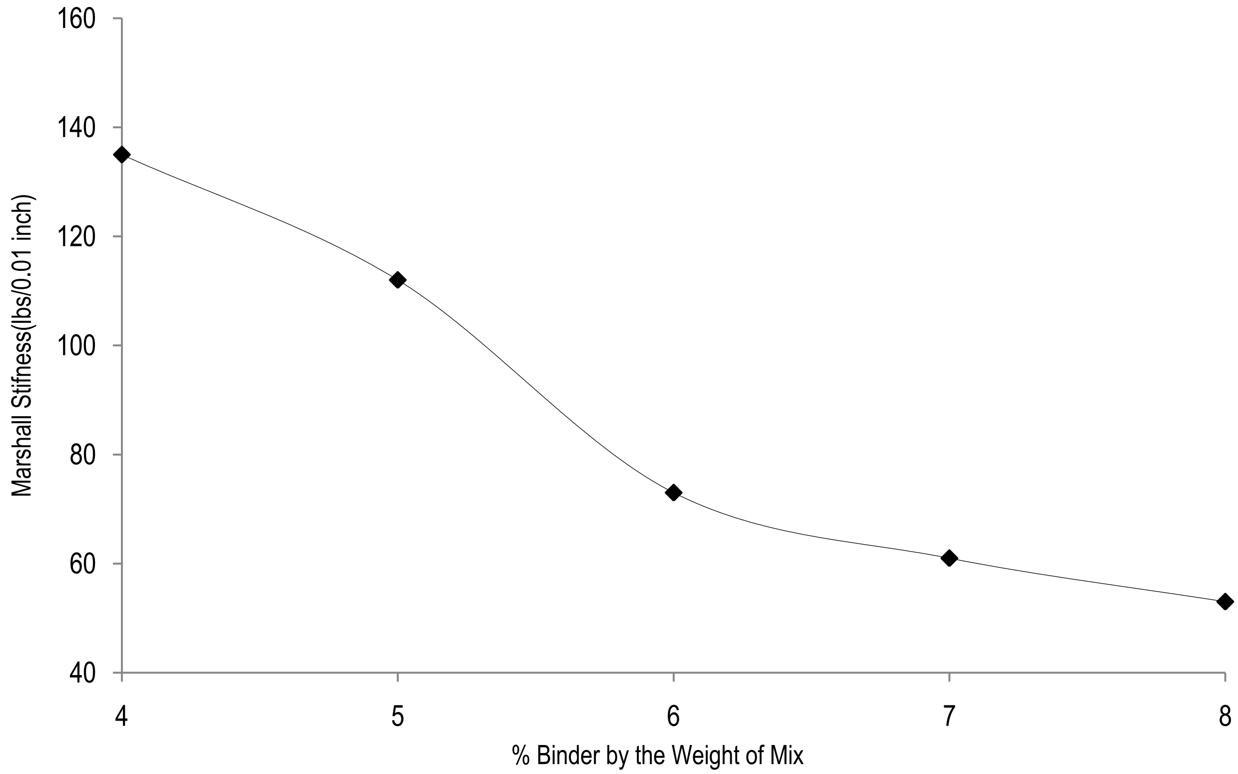

Fig 1.8.7 ; Marshall Stiffness Vs % Bitumen Content

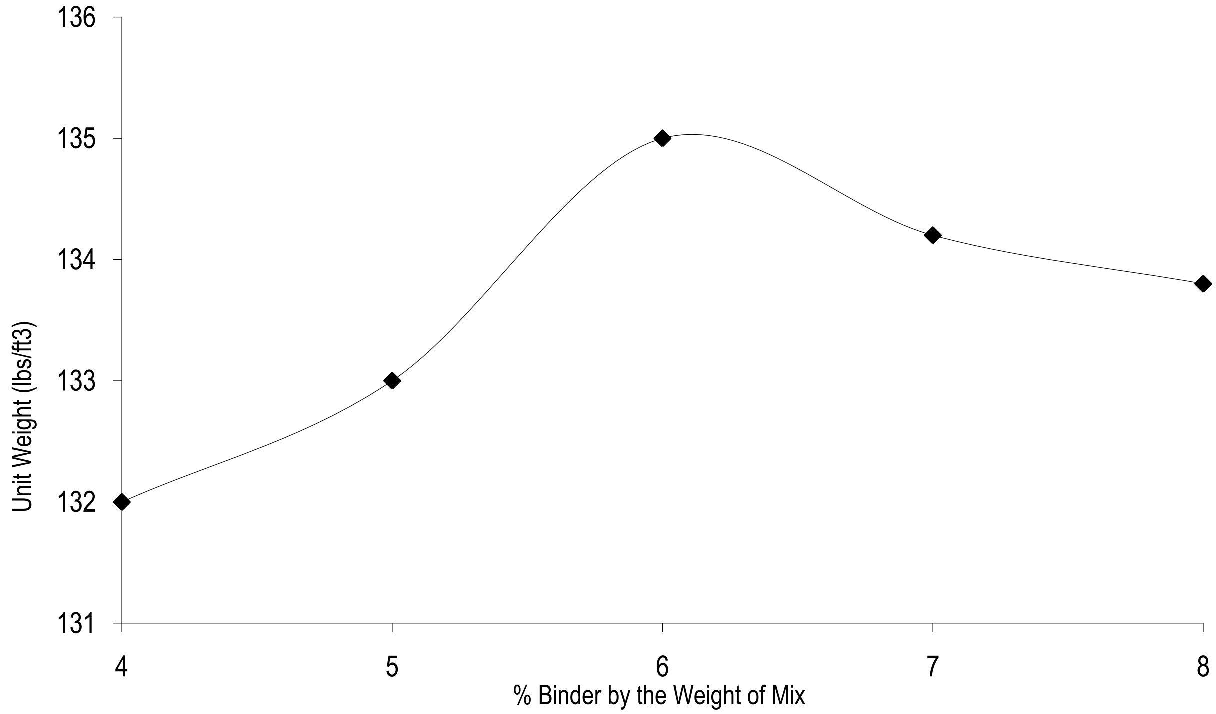

Fig 1.9.1 ; Unit Weight Vs % Bitumen Content

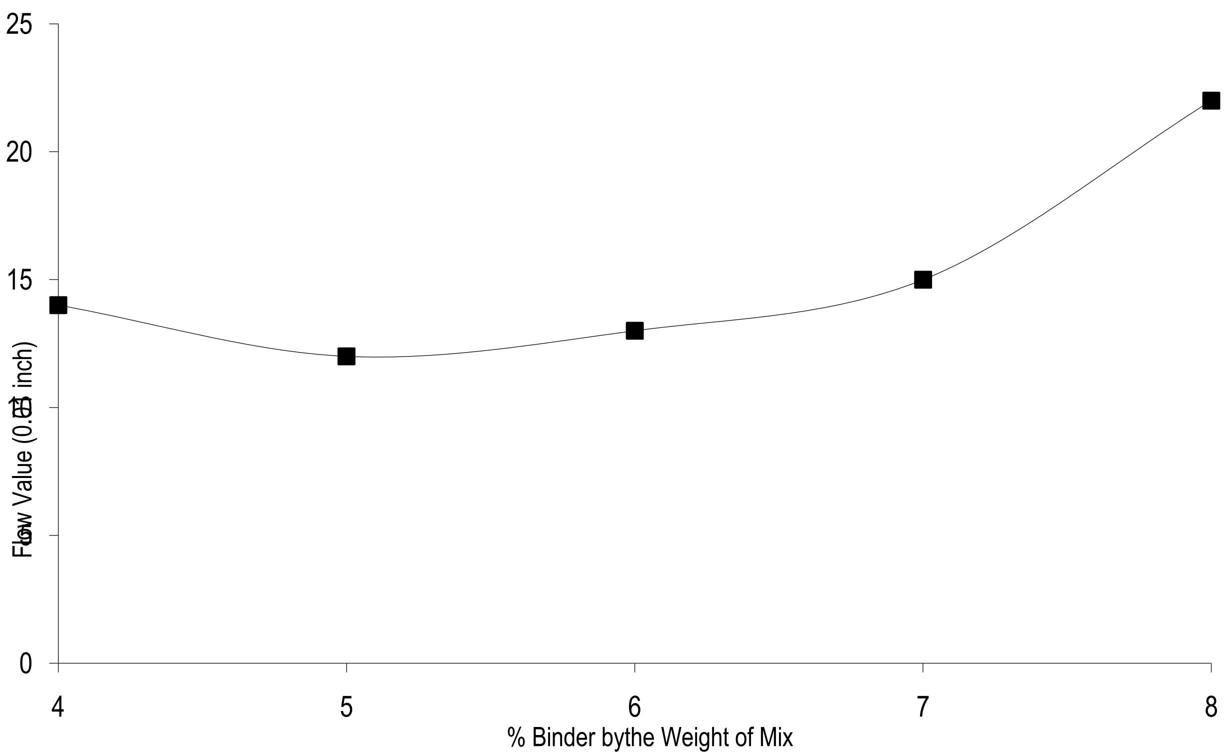

Fig 1.9.2 ; Flow Value Vs % Bitumen Content

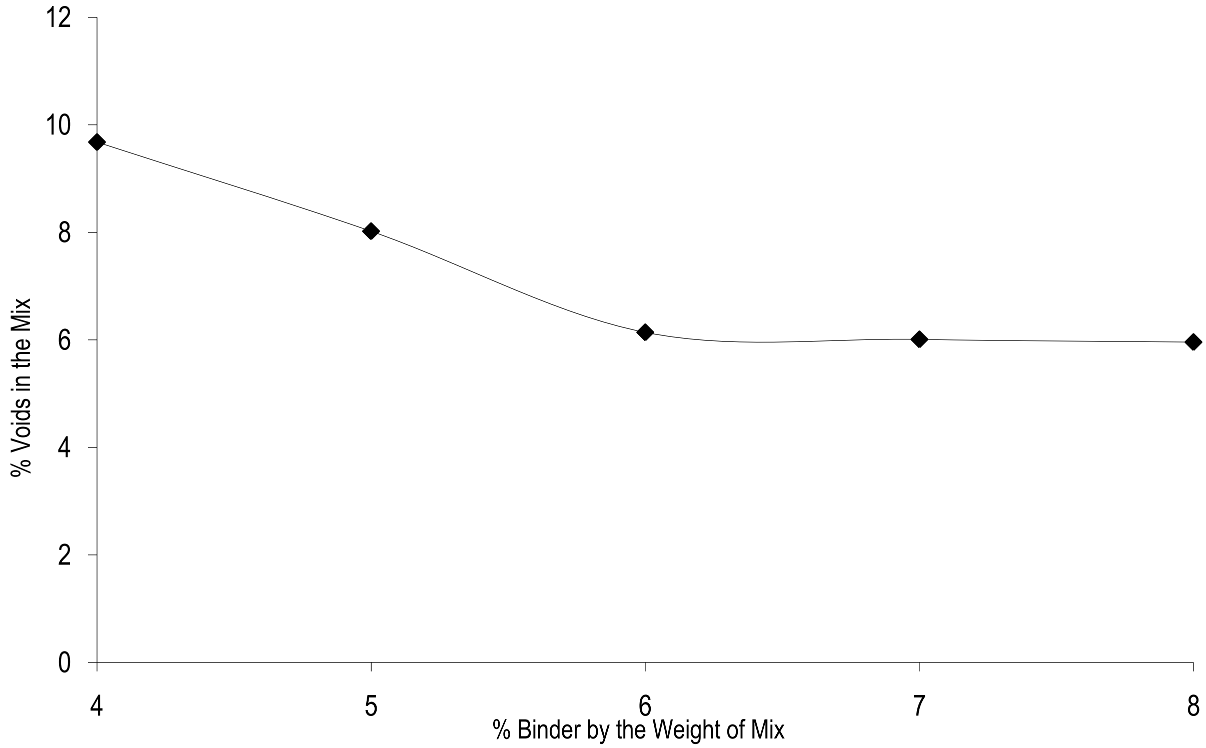

Fig 1.9.3 ; % Voids in the Mix Vs % Bitumen Content

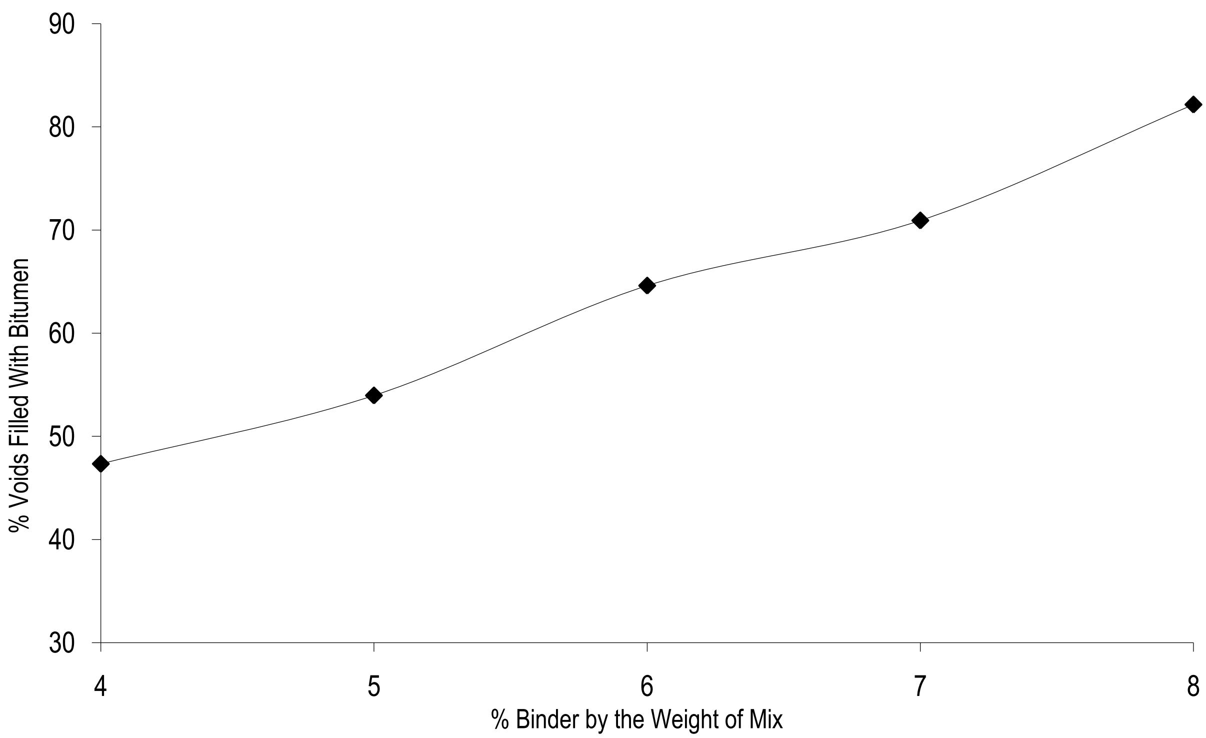

Fig 1.9.4 ; % Voids Filled With Bitumen Vs % Bitumen Content

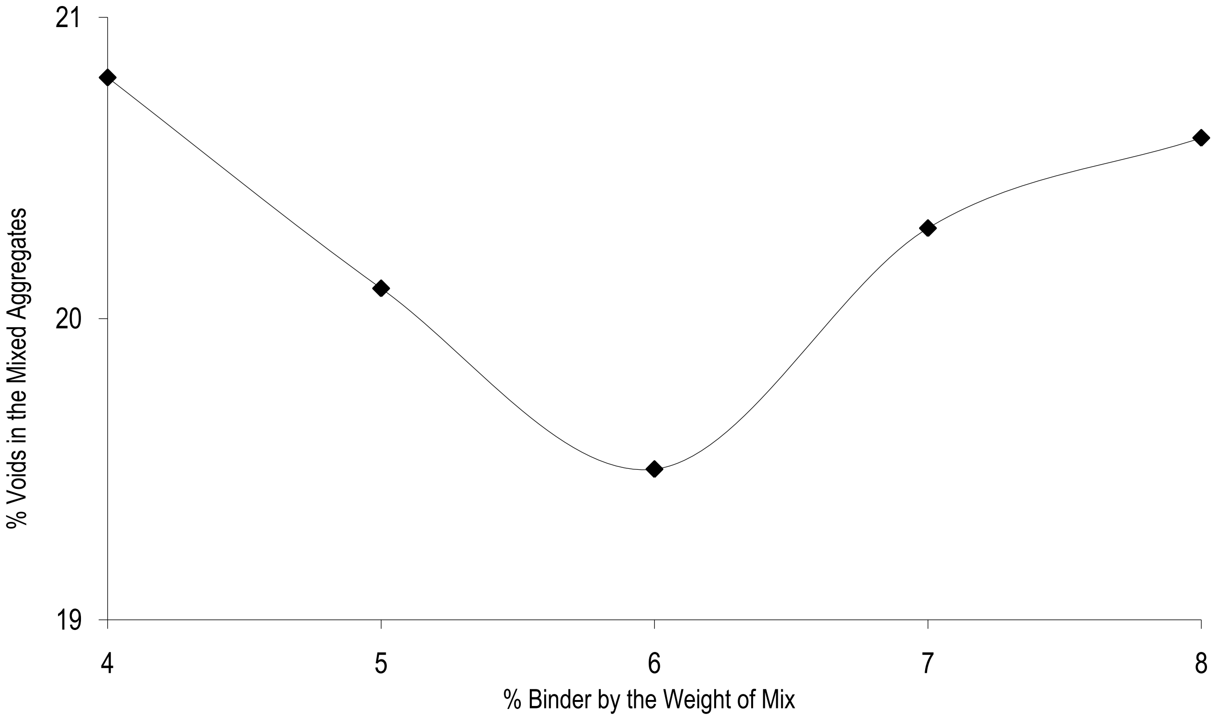

Fig 1.9.5 ; % Voids in the Mixed Aggregates Vs % Bitumen Content

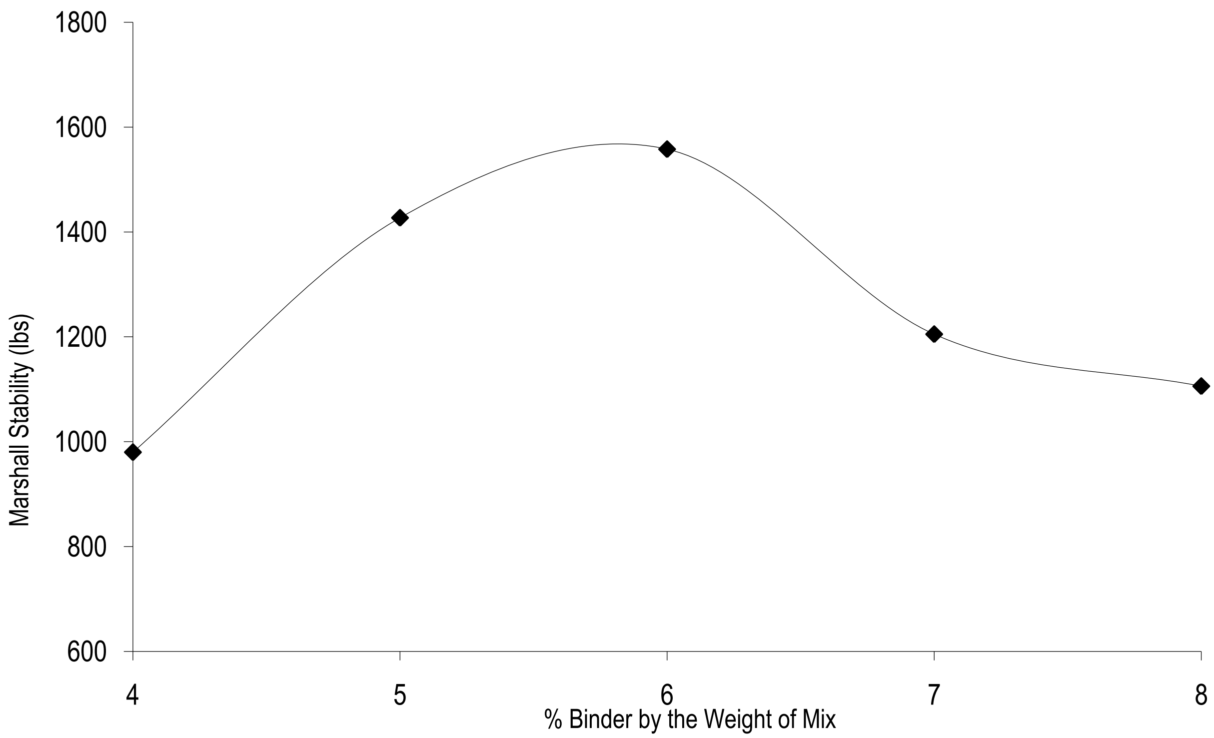

Fig 1.9.6 ; Marshall Stability Vs % Bitumen Content

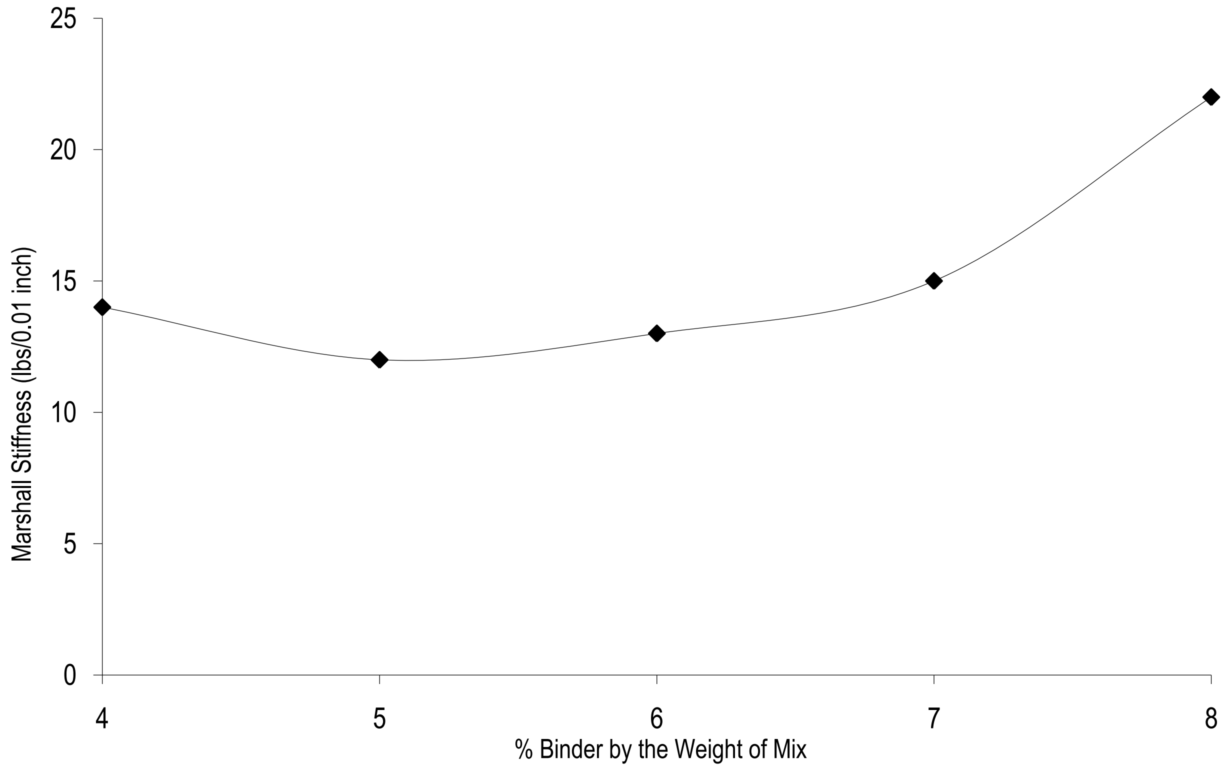

Fig 1.9.7 ; Marshall Stiffness Vs % Bitumen Content

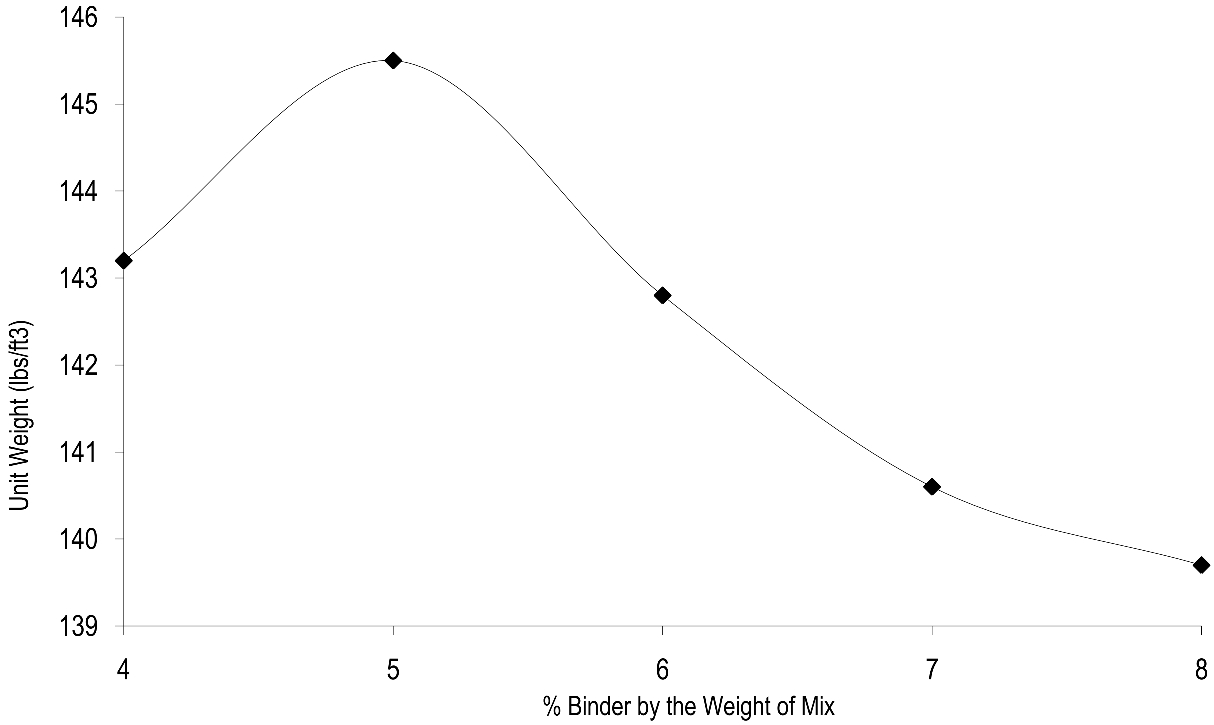

Fig 1.10.1 ; Unit Weight Vs % Bitumen Content

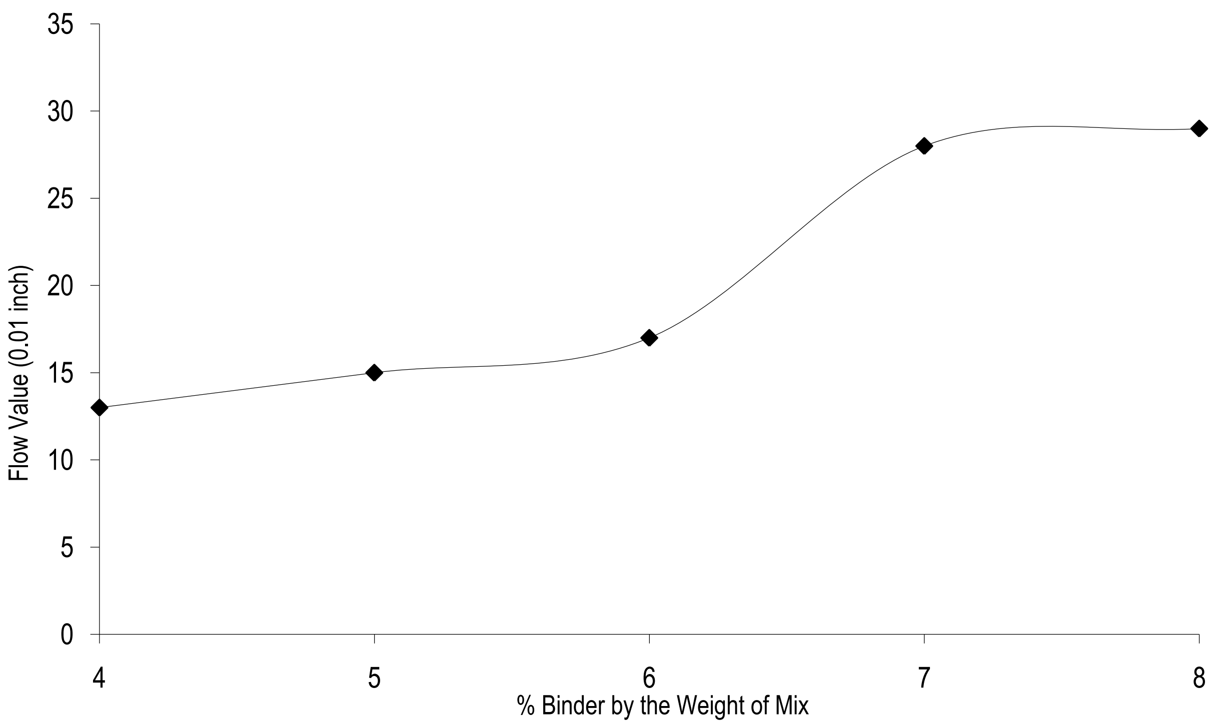

Fig 1.10.2 ; Flow Value Vs % Bitumen Content

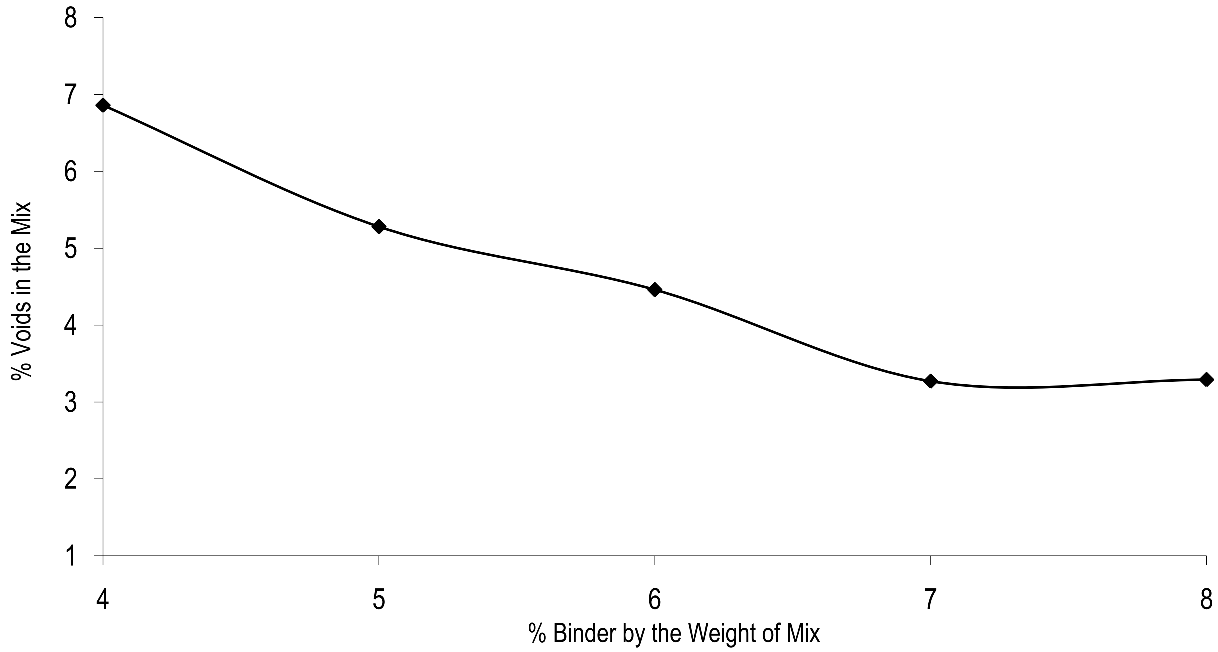

Fig 1.10.3 ; % Voids in the Mix Vs % Bitumen Content

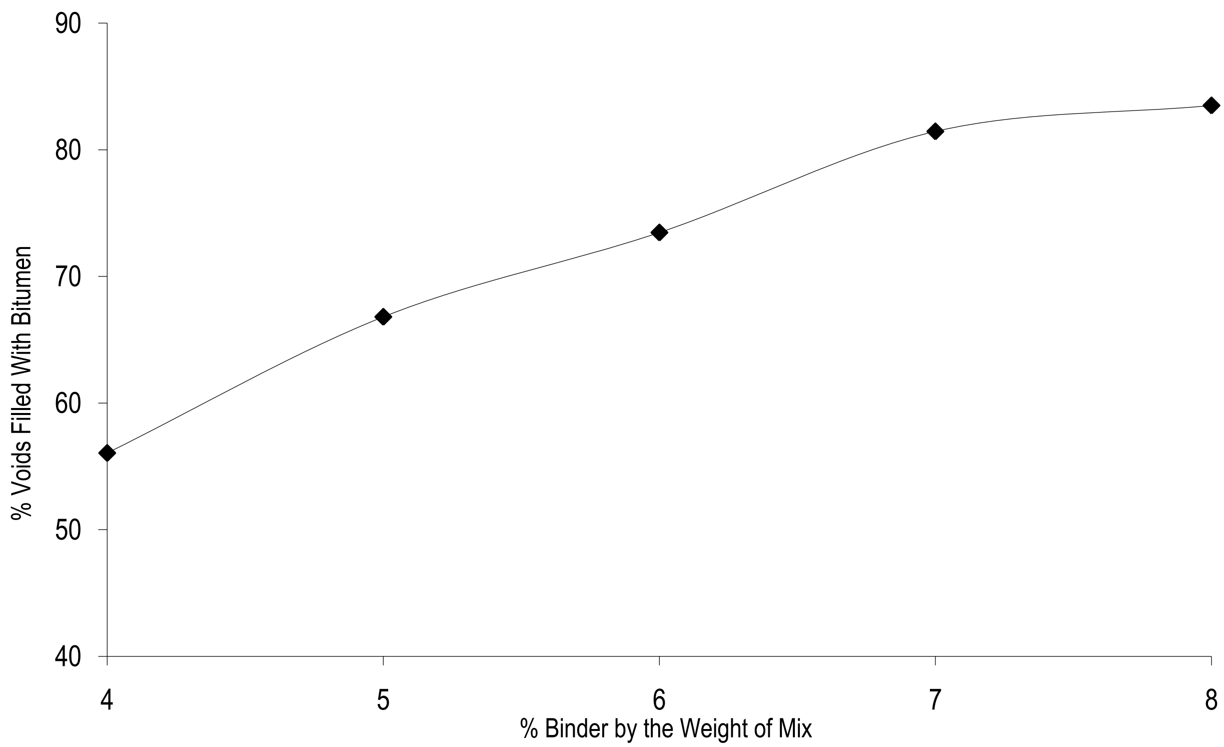

Fig 1.10.4 ; % Voids Filled With Bitumen Vs % Bitumen Content

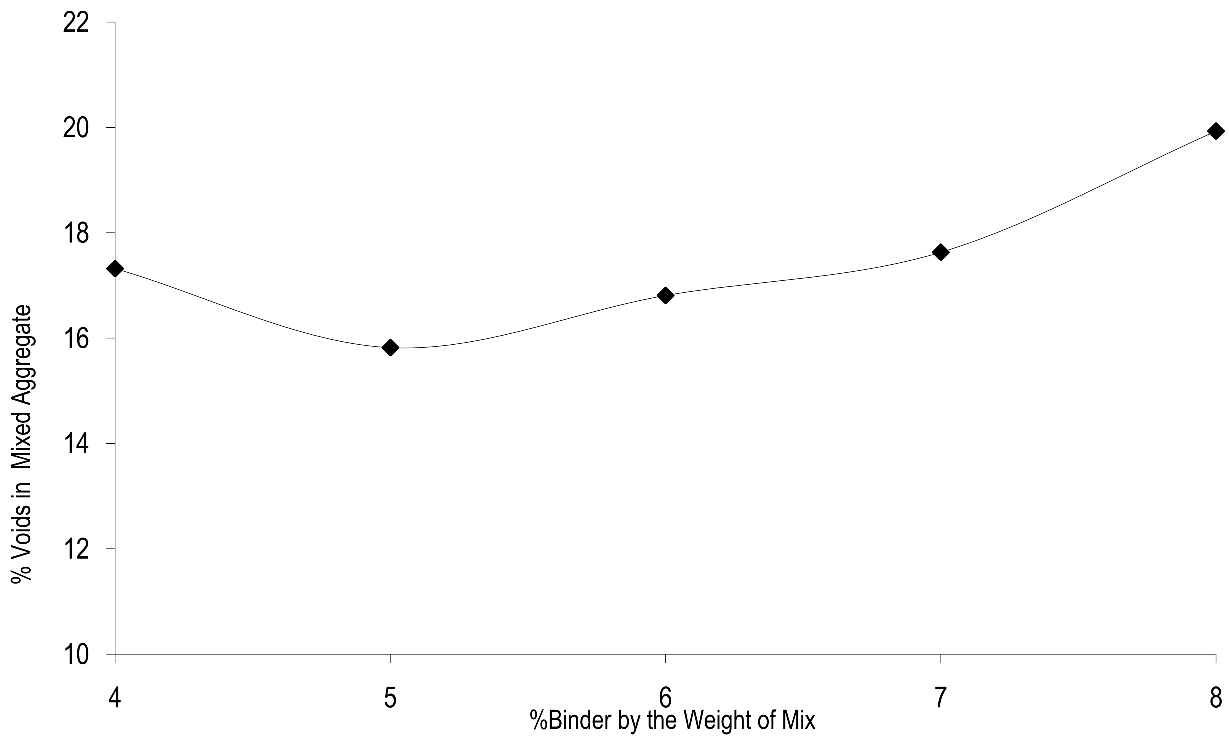

Fig 1.10.5 ; % Voids in the Mixed Aggregates Vs % Bitumen Content

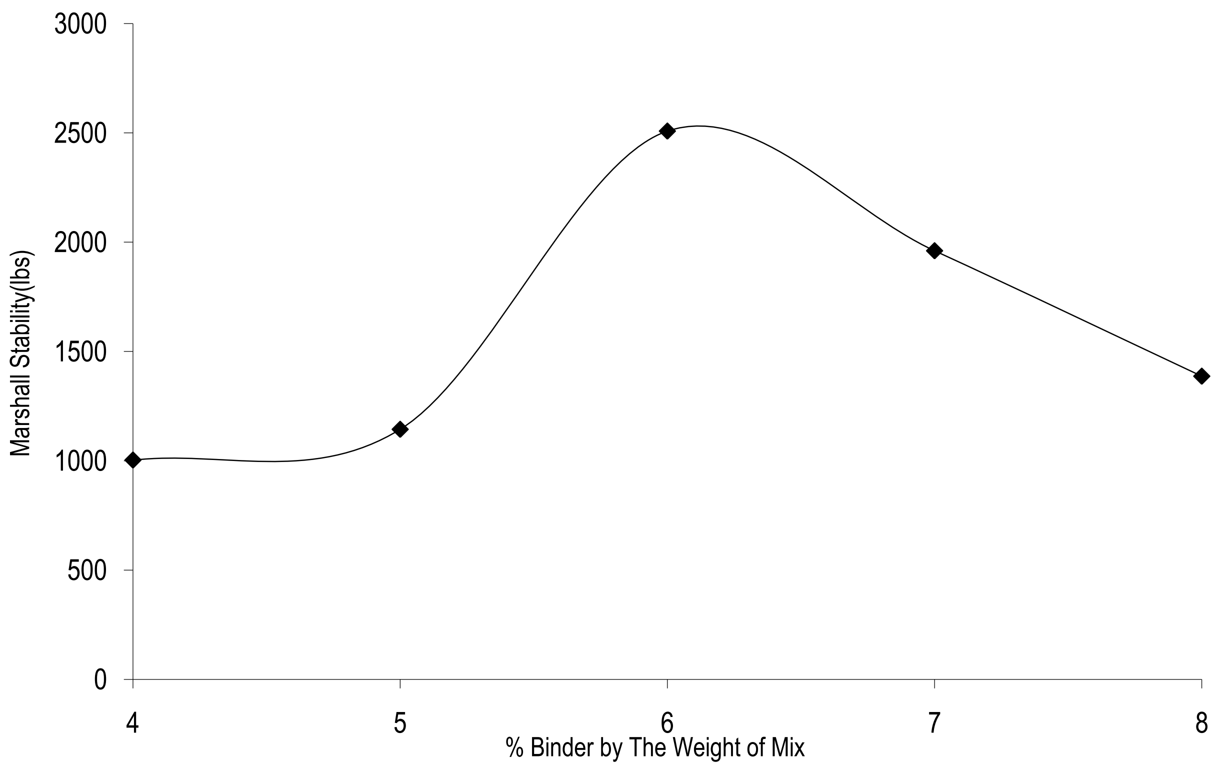

Fig 1.10.6;Marshall Stabilty Vs % Bitumen Content

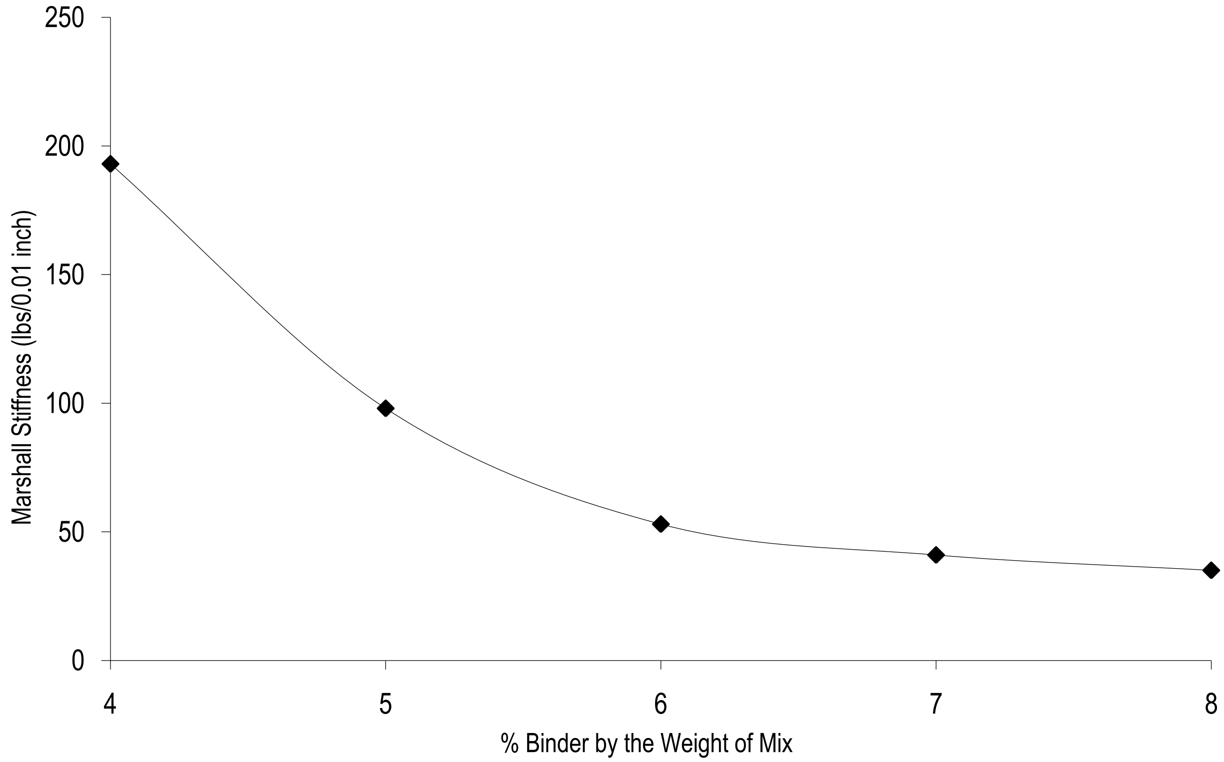

Fig 1.10.7 ; Marshall Stiffness Vs % Bitumen Content

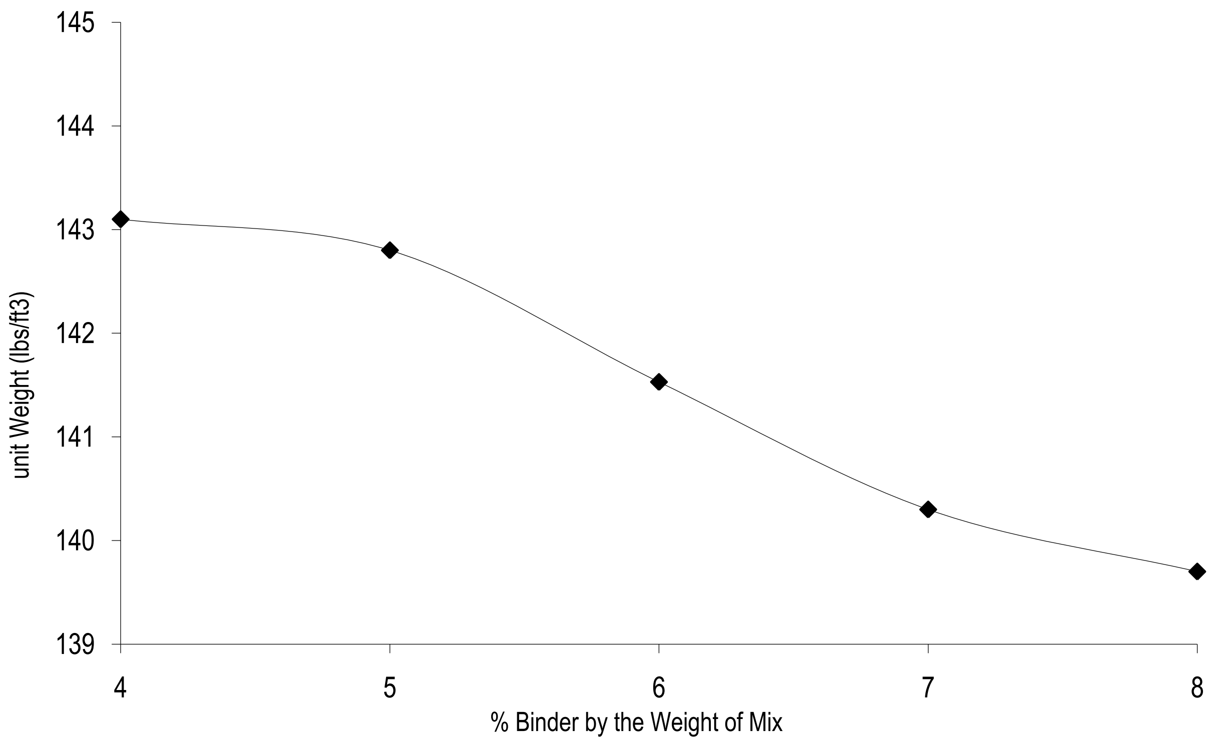

Fig 1.11.1 ; Unit Weight Vs % Bitumen Content

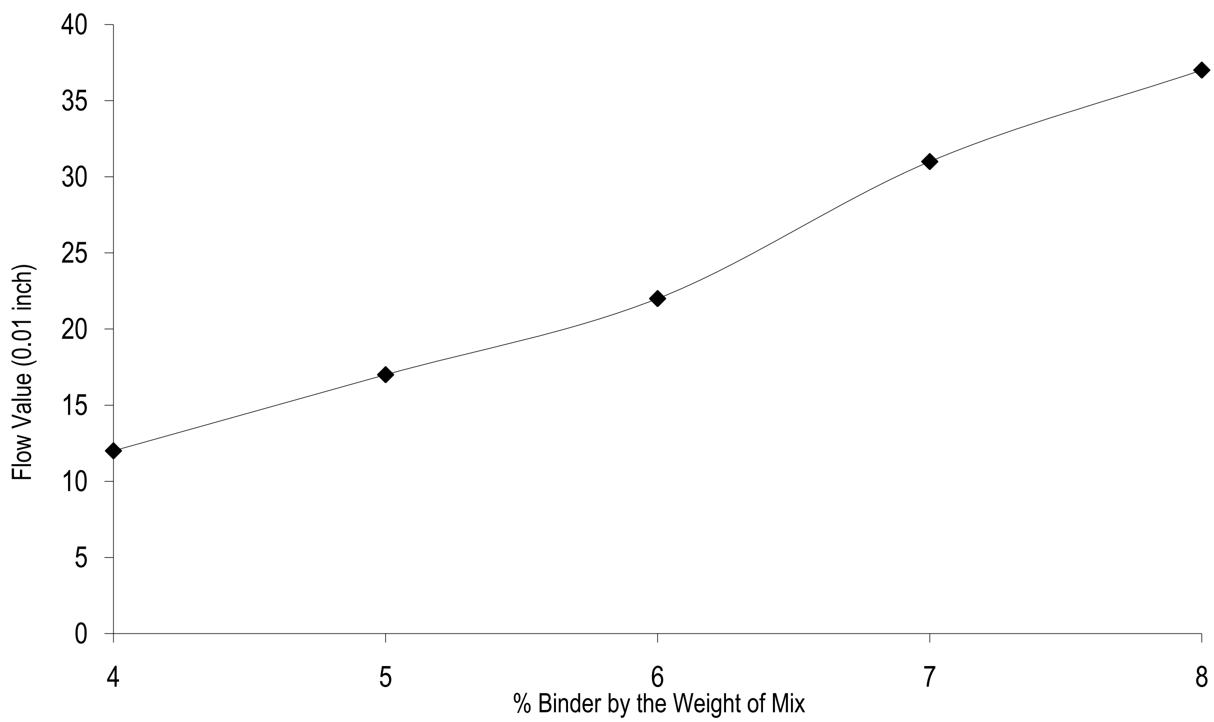

Fig 1.11.2 ; Flow Value Vs % Bitumen Content

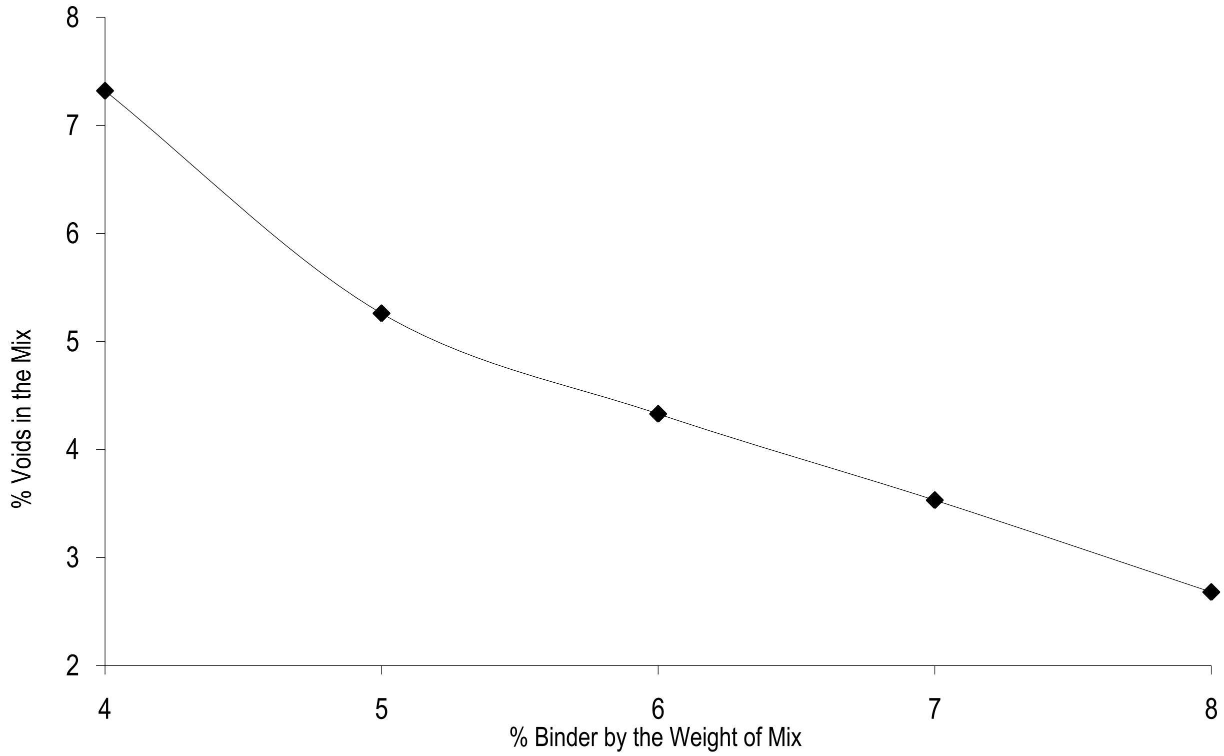

Fig 1.11.3 ; % Voids in the Mix Vs % Bitumen Content

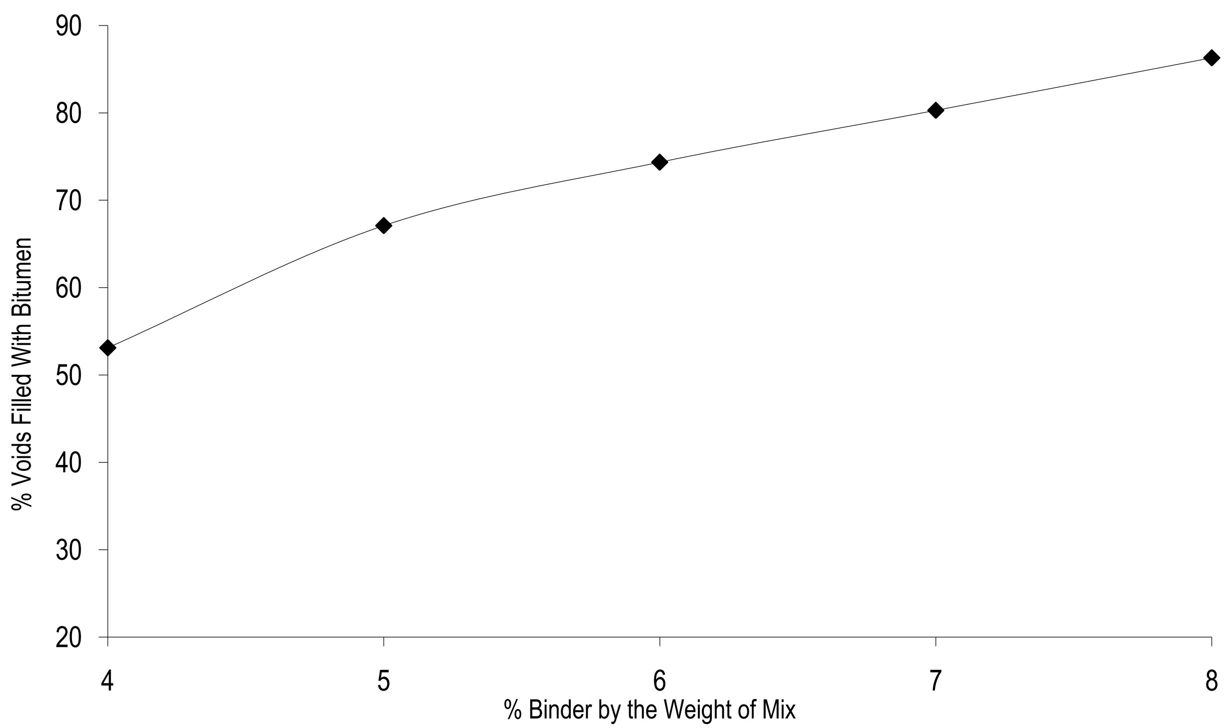

Fig 1.11.4 ; % Voids Filled With Bitumen Vs % Bitumen Content

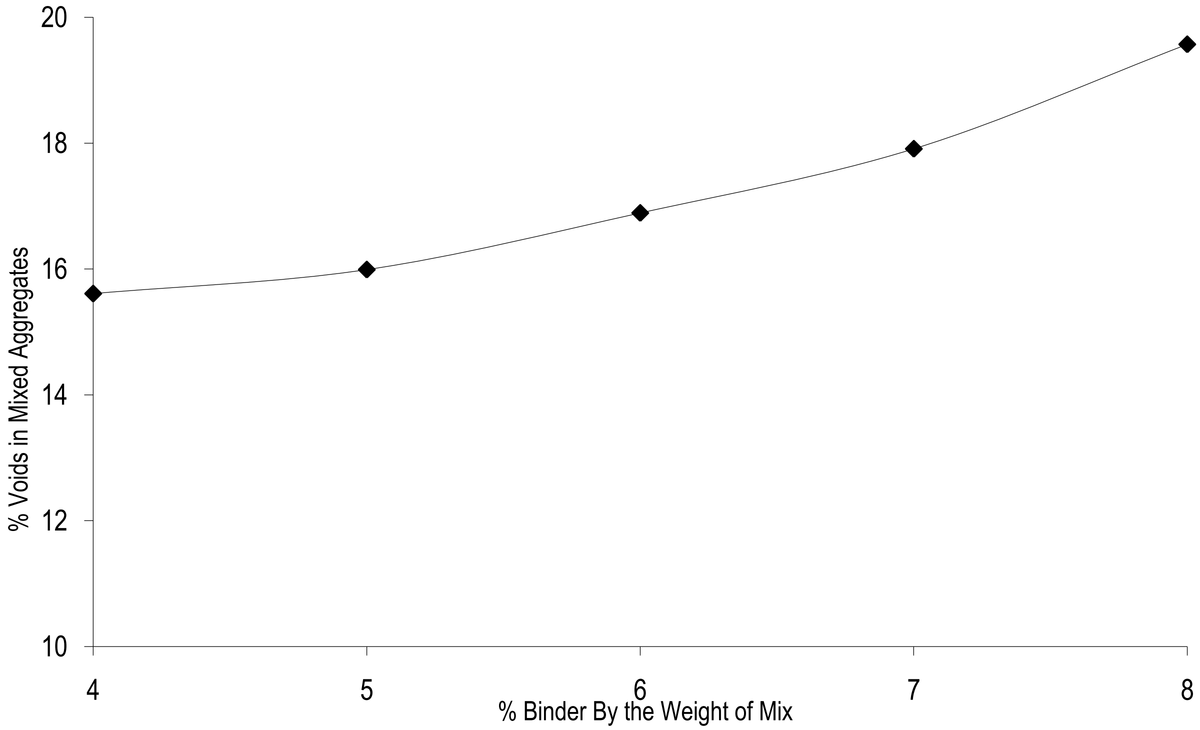

Fig 1.11.5 ; % Voids in Mixed Aggregates Vs % Bitumen Content

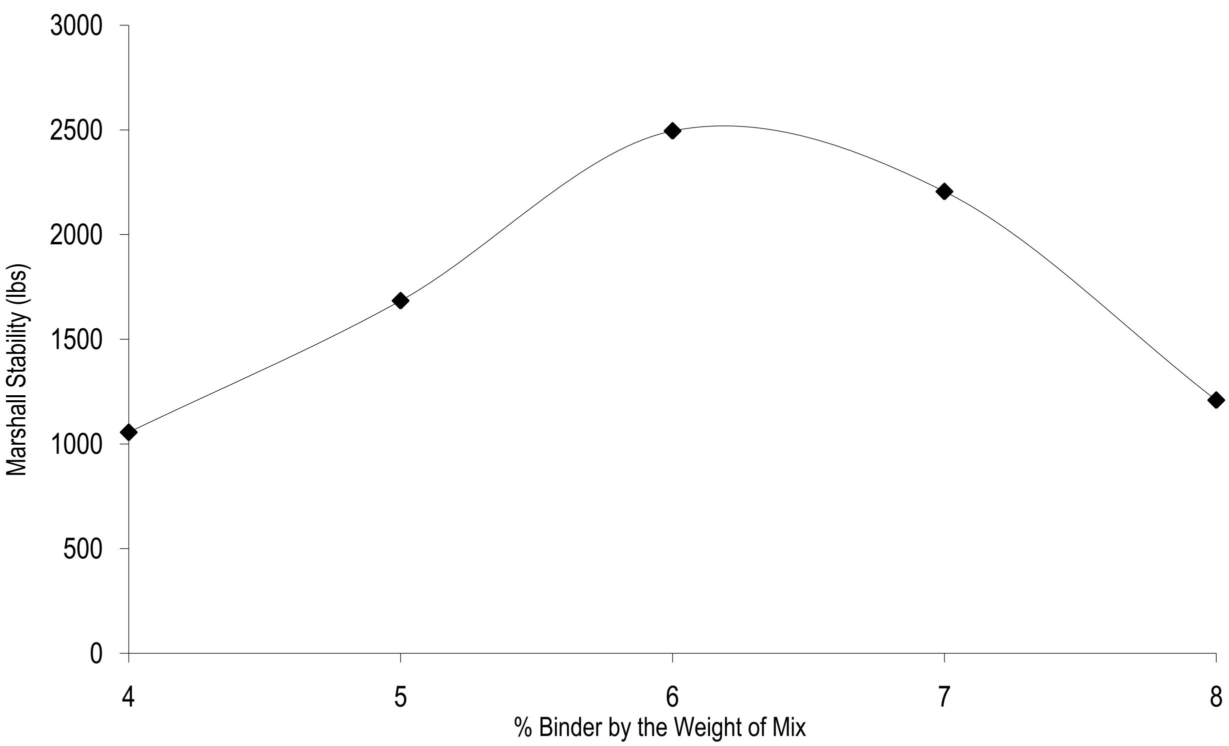

Fig 1.11.6 ; Marshall Stability Vs % Bitumen Content

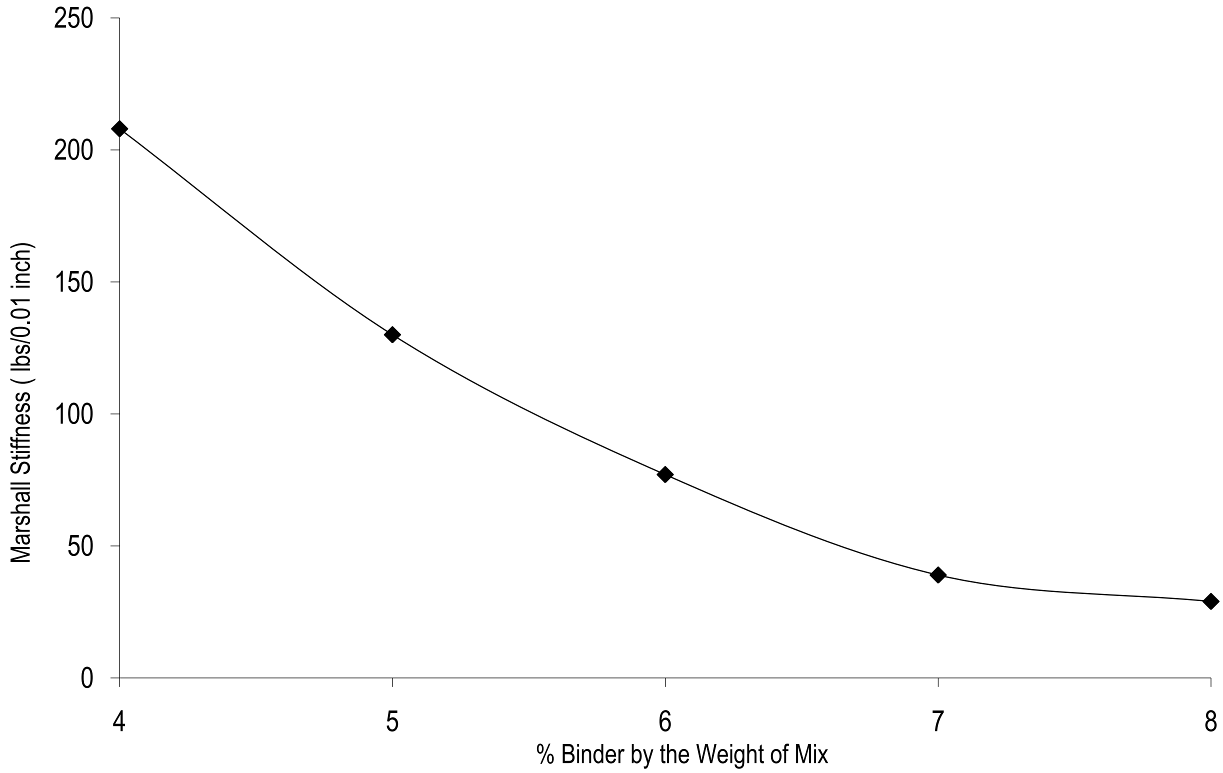

Fig 1.11.7 ; Marshall Stiffness Vs % Bitumen Content

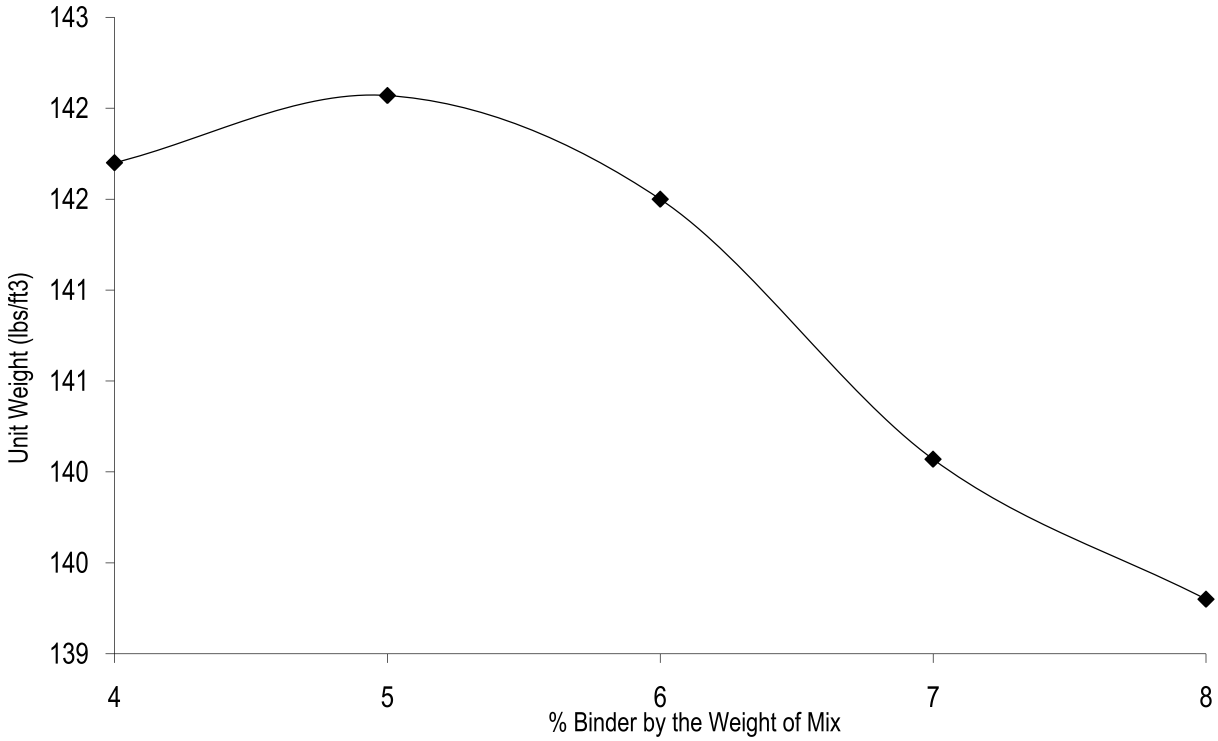

Fig 1.12.1 ; Unit Weight Vs % Bitumen Content

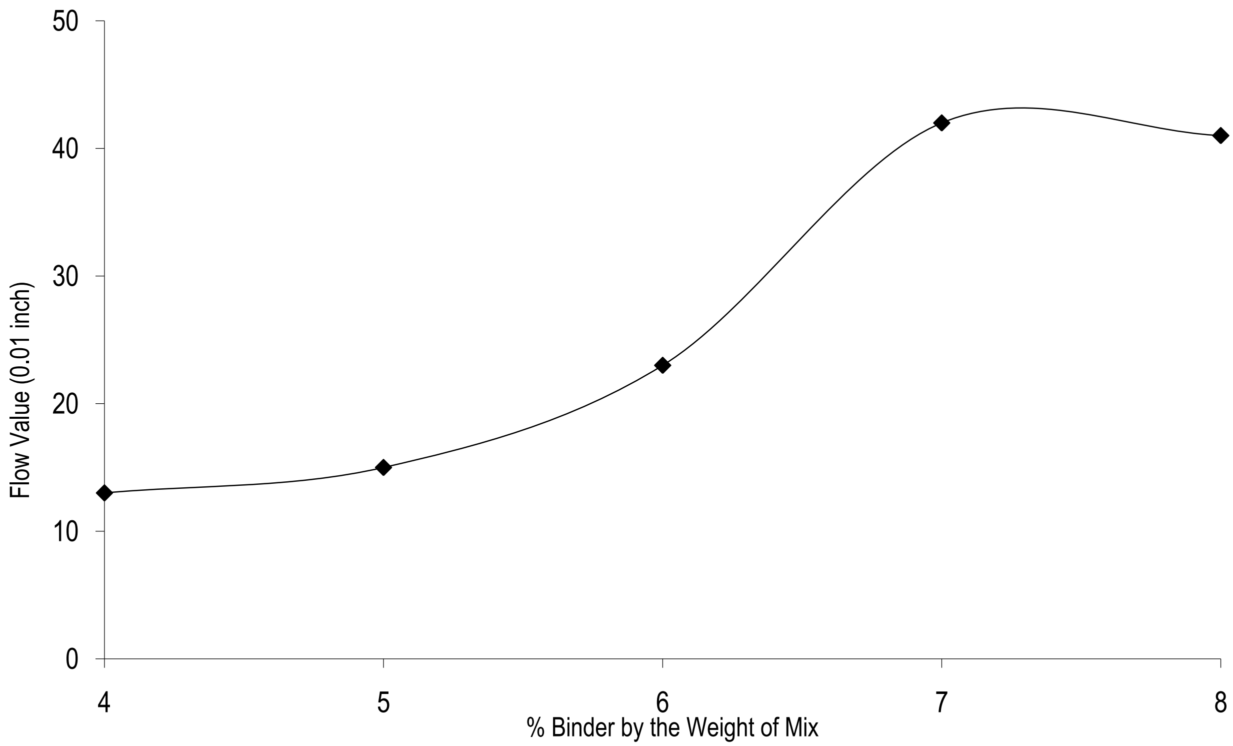

Fig 1.12.2 ; Flow Value Vs % Bitumen Content

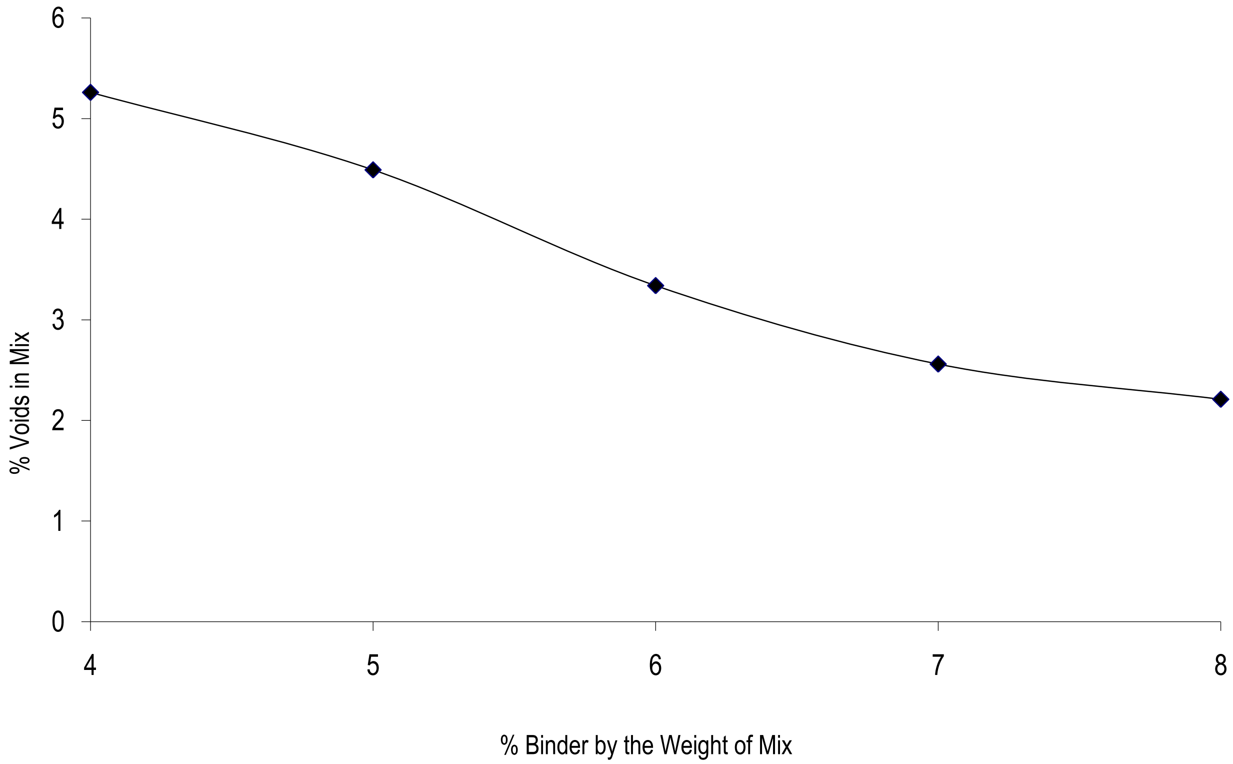

Fig 1.12.3 ; % Voids in the Mix Vs % Bitumen Content

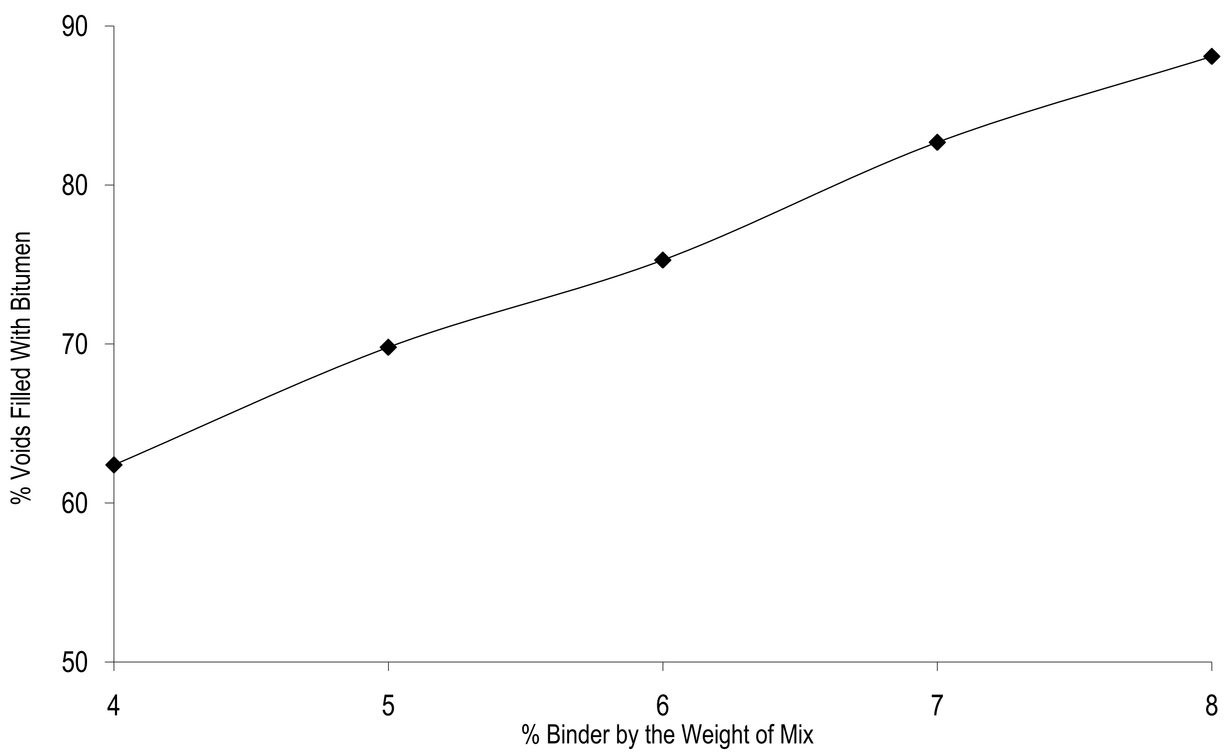

Fig 1.12.4 % Voids Filled With Bitumen Vs % Bitumen Content

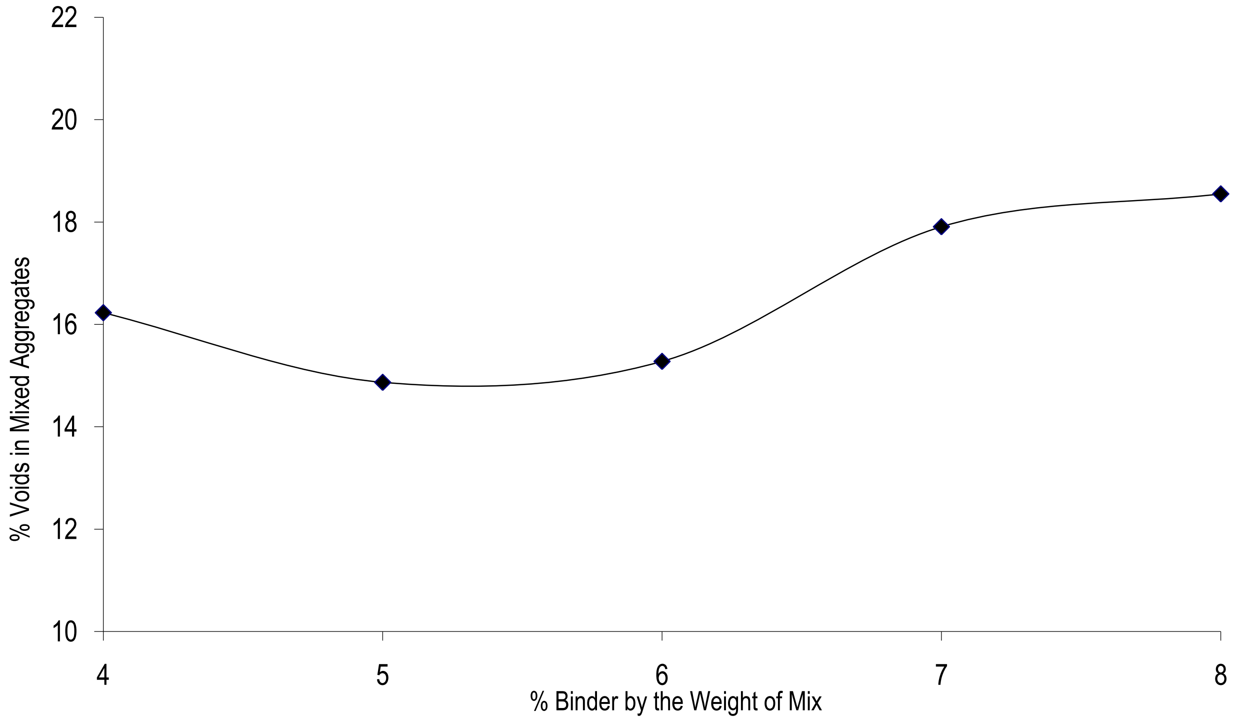

Fig 1.12.5 % Voids in the Mixed Aggregates Vs % Bitumen Content

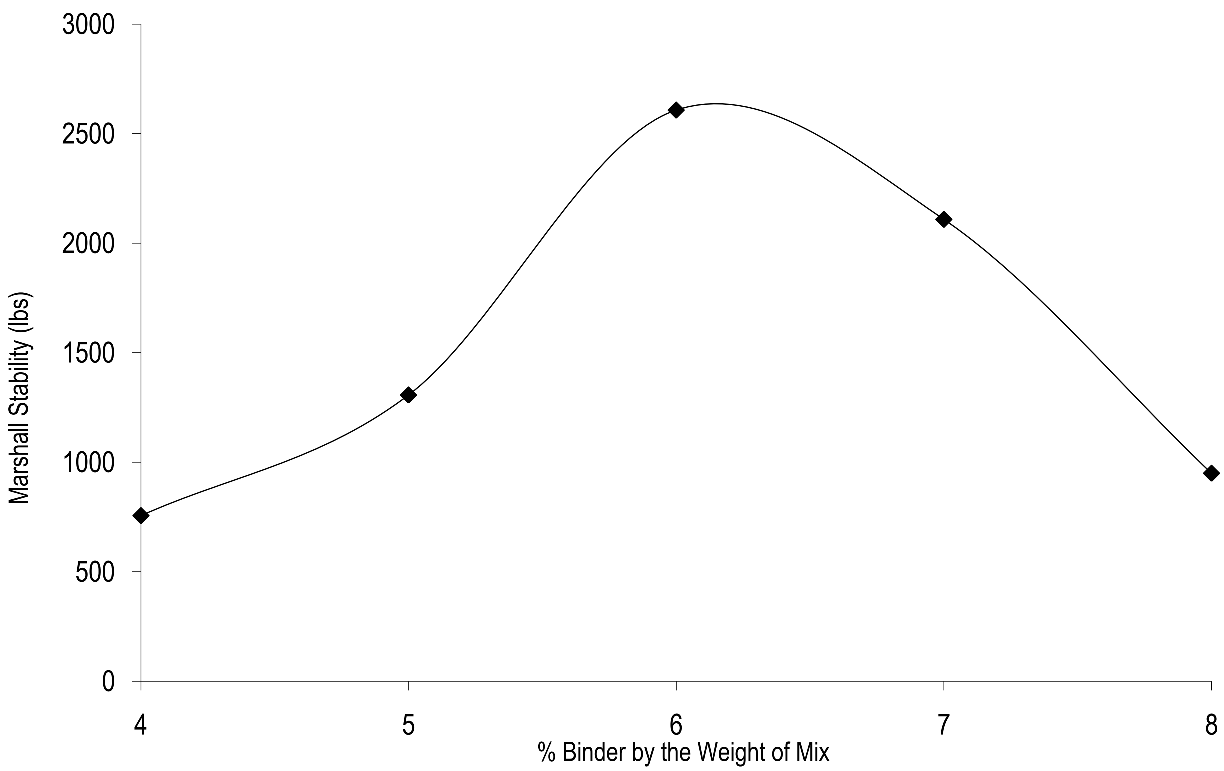

Fig 1.12.6 ; Marshall Stability Vs % Bitumen Content

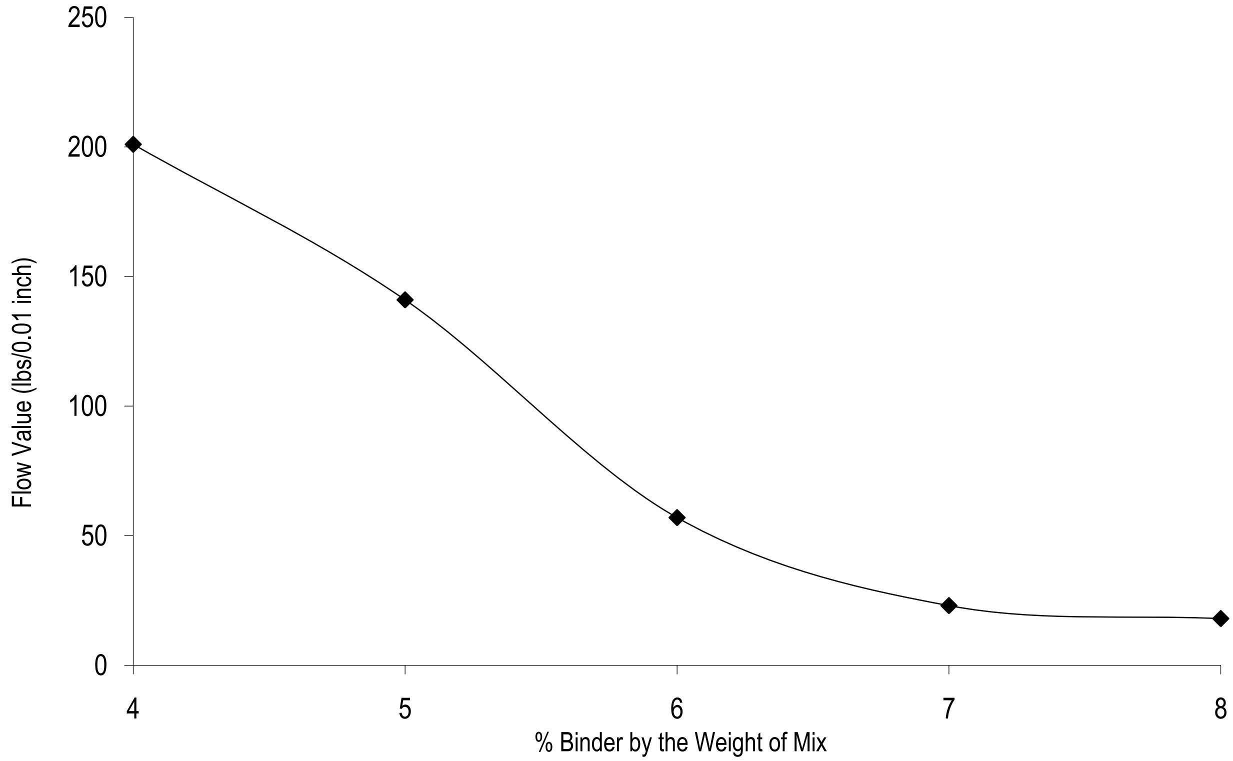

Fig 1.12.7 ; Flow Value Vs % Bitumen Content

APPENDIX 2

Table 1.1; Permanent Deformation Parameters for Samples Made Using 60/70 Penetration Grade Bitumen and Tested at 20^0C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	4	0.92	0.94	1.37	43.0	40.0	52.4	0.123	0.146	0.179	0.123	0.146	0.179	0.511	0.510	0.510
	5	0.62	1.18	1.74	29.0	38.7	44.6	0.200	0.283	0.154	0.200	0.283	0.154	0.510	0.508	0.508
	6	0.70	1.53	1.75	27.6	50.3	67.7	0.129	0.185	0.262	0.129	0.185	0.262	0.508	0.506	0.508
	7	0.75	1.41	2.06	25.1	53.3	67.2	0.247	0.235	0.313	0.247	0.235	0.313	0.508	0.508	0.508
	8	0.79	1.30	1.60	36.4	76.9	94.4	0.115	0.353	0.446	0.115	0.353	0.446	0.510	0.508	0.508
2	4	0.83	1.21	1.66	26.7	42.4	66.7	0.070	0.118	0.543	0.070	0.118	0.543	0.511	0.510	0.508
	5	1.17	1.30	2.37	47.4	49.3	96.2	0.126	0.182	0.269	0.126	0.182	0.269	0.511	0.510	0.511
	6	0.77	0.93	1.10	33.9	47.7	53.9	0.104	0.308	0.255	0.104	0.308	0.255	0.510	0.508	0.510
	7	1.21	1.62	2.39	57.9	60.5	66.7	0.160	0.200	0.273	0.160	0.200	0.273	0.510	0.506	0.506
	8	0.55	1.52	1.91	16.2	63.3	77.6	0.057	0.425	0.582	0.057	0.425	0.582	0.508	0.508	0.508
3	4	1.26	1.56	1.96	50.0	55.7	63.6	0.137	0.176	0.193	0.137	0.176	0.193	0.510	0.510	0.510
	5	1.37	1.53	1.66	53.3	65.1	67.7	0.165	0.494	0.215	0.165	0.494	0.215	0.510	0.510	0.508
	6	1.32	1.48	1.87	38.5	68.3	84.1	0.178	0.250	0.431	0.178	0.250	0.431	0.508	0.508	0.508
	7	1.66	1.90	2.39	41.0	42.2	45.5	0.446	0.267	0.159	0.446	0.267	0.159	0.508	0.506	0.508
	8	0.96	1.41	2.0	20.0	37.3	57.4	0.07	0.173	0.369	0.07	0.173	0.369	0.506	0.506	0.508
4	4	1.12	1.21	1.54	70.0	72.6	87.8	0.364	0.600	0.965	0.364	0.600	0.965	0.510	0.510	0.510
	5	1.15	1.52	1.94	42.7	43.0	47.2	0.140	0.155	0.154	0.140	0.155	0.154	0.510	0.510	0.508
	6	1.25	1.61	2.44	80.0	57.8	86.4	0.400	0.483	0.568	0.400	0.483	0.568	0.510	0.506	0.506
	7	0.72	1.23	1.87	32.9	44.4	49.2	0.106	0.233	0.150	0.106	0.233	0.150	0.510	0.508	0.508
	8	1.16	1.22	1.85	36.3	14.8	22.6	0.200	0.044	0.077	0.200	0.044	0.077	0.508	0.508	0.508

Table 1.2; Permanent Deformation Parameters for Samples Made Using 60/70 Penetration Grade Bitumen and Tested at 30^0C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	4	0.49	0.50	0.74	23.3	21.9	26.7	0.073	0.071	0.071	0.073	0.071	0.071	0.511	0.510	0.510
	5	0.34	0.64	0.94	17.8	20.0	23.6	0.100	0.121	0.062	0.100	0.121	0.062	0.510	0.508	0.508
	6	0.38	0.83	0.95	15.7	26.7	36.9	0.088	0.085	0.138	0.088	0.085	0.138	0.508	0.506	0.508
	7	0.41	0.76	1.11	13.3	28.7	37.2	0.067	0.115	0.167	0.067	0.115	0.167	0.508	0.508	0.508
	8	0.43	0.71	0.87	20.0	41.6	50.3	0.100	0.176	0.210	0.100	0.176	0.210	0.510	0.508	0.508
2	4	0.62	0.91	1.25	20.0	31.8	48.9	0.047	0.086	0.367	0.047	0.086	0.367	0.511	0.510	0.508
	5	0.88	0.98	1.78	33.8	36.5	72.2	0.108	0.139	0.233	0.108	0.139	0.233	0.511	0.510	0.511
	6	0.58	0.70	0.82	24.3	35.0	41.4	0.082	0.250	0.286	0.082	0.250	0.286	0.510	0.508	0.510
	7	0.91	1.22	1.79	42.1	45.7	58.7	0.148	0.314	0.400	0.148	0.314	0.400	0.510	0.506	0.506
	8	0.42	1.14	1.43	11.6	44.8	56.7	0.044	0.318	0.338	0.044	0.318	0.338	0.508	0.508	0.508
3	4	0.87	1.08	1.35	35.9	38.8	40.8	0.108	0.109	0.125	0.108	0.109	0.125	0.510	0.510	0.510
	5	0.95	1.06	1.14	37.5	45.1	45.6	0.114	0.320	0.142	0.114	0.320	0.142	0.510	0.510	0.508
	6	0.91	1.02	1.29	26.7	28.7	61.3	0.123	0.080	0.200	0.123	0.080	0.200	0.508	0.508	0.508
	7	1.15	1.33	1.65	26.7	29.5	32.0	0.233	0.200	0.092	0.233	0.200	0.092	0.508	0.506	0.508
	8	0.66	0.97	1.38	14.6	24.0	38.3	0.050	0.090	0.250	0.050	0.090	0.250	0.506	0.506	0.508
4	4	0.70	0.75	0.96	42.5	44.2	59.4	0.171	0.250	0.164	0.171	0.250	0.164	0.510	0.510	0.510
	5	0.71	0.94	1.20	26.7	24.6	23.7	0.071	0.077	0.056	0.071	0.077	0.056	0.510	0.510	0.508
	6	0.78	1.00	1.51	48.6	33.3	34.7	0.250	0.127	0.200	0.250	0.127	0.200	0.510	0.506	0.506
	7	0.45	0.76	1.16	20.0	28.2	32.0	0.054	0.123	0.120	0.054	0.123	0.120	0.510	0.508	0.508
	8	0.72	0.76	1.15	22.5	7.5	13.3	0.163	0.025	0.040	0.163	0.025	0.040	0.508	0.508	0.508

Table 1.3; Permanent Deformation Parameters for Samples Made Using 60/70 Penetration Grade Bitumen and Tested at 40^0C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	4	0.27	0.28	0.41	13.0	15.7	14.3	0.075	0.076	0.048	0.075	0.076	0.048	0.511	0.510	0.510
	5	0.19	0.35	0.52	8.5	11.2	13.3	0.014	0.063	0.048	0.014	0.063	0.048	0.510	0.508	0.508
	6	0.21	0.45	0.52	8.6	14.9	21.3	0.118	0.058	0.100	0.118	0.058	0.100	0.508	0.506	0.508
	7	0.22	0.42	0.61	7.6	15.9	20.7	0.029	0.077	0.110	0.029	0.077	0.110	0.508	0.508	0.508
	8	0.34	0.39	0.48	11.8	22.4	27.9	0.041	0.091	0.140	0.041	0.091	0.140	0.510	0.508	0.508
2	4	0.34	0.50	0.69	10.5	17.6	26.7	0.029	0.054	0.133	0.029	0.054	0.133	0.511	0.510	0.508
	5	0.48	0.54	0.98	20.0	20.6	37.5	0.064	0.100	0.120	0.064	0.100	0.120	0.511	0.510	0.511
	6	0.32	0.39	0.45	13.9	19.7	22.6	0.041	0.146	0.162	0.041	0.146	0.162	0.510	0.508	0.510
	7	0.50	0.67	0.99	23.0	26.7	28.5	0.076	0.082	0.109	0.076	0.082	0.109	0.510	0.506	0.506
	8	0.23	0.63	0.79	6.4	25.9	32.0	0.023	0.112	0.233	0.023	0.112	0.233	0.508	0.508	0.508
3	4	0.48	0.59	0.74	18.8	20.4	24.3	0.047	0.059	0.071	0.047	0.059	0.071	0.510	0.510	0.510
	5	0.52	0.58	0.63	20.0	24.2	25.6	0.064	0.125	0.082	0.064	0.125	0.082	0.510	0.510	0.508
	6	0.50	0.56	0.71	13.3	17.1	32.5	0.052	0.050	0.200	0.052	0.050	0.200	0.508	0.508	0.508
	7	0.63	0.73	0.91	14.9	15.9	19.5	0.165	0.075	0.062	0.165	0.075	0.062	0.508	0.506	0.508
	8	0.37	0.54	0.76	8.0	14.5	21.7	0.027	0.082	0.150	0.027	0.082	0.150	0.506	0.506	0.508
4	4	0.42	0.45	0.57	25.3	26.7	33.7	0.116	0.115	0.176	0.116	0.115	0.176	0.510	0.510	0.510
	5	0.43	0.57	0.72	16.4	16.2	15.8	0.069	0.050	0.050	0.069	0.050	0.050	0.510	0.510	0.508
	6	0.47	0.60	0.91	29.2	20.8	30.0	0.150	0.113	0.157	0.150	0.113	0.157	0.510	0.506	0.506
	7	0.27	0.46	0.69	12.5	17.4	19.2	0.048	0.077	0.063	0.048	0.077	0.063	0.510	0.508	0.508
	8	0.43	0.46	0.69	14.4	5.1	7.5	0.123	0.015	0.025	0.123	0.015	0.025	0.508	0.508	0.508

Table 1.4; Permanent Deformation Parameters for Samples Made Using 60/70 Penetration Grade Bitumen and Tested at 50^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	4	0.21	0.21	0.31	9.4	9.5	11.3	0.018	0.030	0.031	0.018	0.030	0.031	0.511	0.510	0.510
	5	0.14	0.27	0.39	6.7	11.5	11.3	0.046	0.059	0.054	0.046	0.059	0.054	0.510	0.508	0.508
	6	0.16	0.35	0.40	6.4	10.8	15.5	0.026	0.050	0.060	0.026	0.050	0.060	0.508	0.506	0.508
	7	0.17	0.32	0.46	5.6	15.6	14.9	0.072	0.050	0.008	0.072	0.050	0.008	0.508	0.508	0.508
	8	0.18	0.29	0.36	8.3	17.3	21.0	0.031	0.065	0.106	0.031	0.065	0.106	0.510	0.508	0.508
2	4	0.24	0.35	0.48	7.6	12.5	20.0	0.024	0.040	0.118	0.024	0.040	0.118	0.511	0.510	0.508
	5	0.34	0.38	0.68	13.3	13.8	27.5	0.042	0.050	0.088	0.042	0.050	0.088	0.511	0.510	0.511
	6	0.23	0.27	0.32	0.142	14.8	16.3	0.029	0.057	0.063	0.029	0.057	0.063	0.510	0.508	0.510
	7	0.35	0.47	0.69	15.8	18.0	20.0	0.044	0.054	0.088	0.044	0.054	0.088	0.510	0.506	0.506
	8	0.16	0.44	0.55	3.9	16.7	21.3	0.040	0.129	0.140	0.040	0.129	0.140	0.508	0.508	0.508
3	4	0.30	0.37	0.47	11.8	13.3	15.4	0.035	0.040	0.046	0.035	0.040	0.046	0.510	0.510	0.510
	5	0.33	0.37	0.40	12.9	15.3	15.9	0.056	0.047	0.046	0.056	0.047	0.046	0.510	0.510	0.508
	6	0.31	0.35	0.45	8.6	11.1	21.1	0.029	0.057	0.100	0.029	0.057	0.100	0.508	0.508	0.508
	7	0.40	0.45	0.57	10.0	10.0	11.3	0.103	0.057	0.040	0.103	0.057	0.040	0.508	0.506	0.508
	8	0.23	0.34	0.48	5.3	8.8	14.0	0.020	0.038	0.103	0.020	0.038	0.103	0.506	0.506	0.508
4	4	0.27	0.28	0.36	16.5	15.9	22.4	0.143	0.119	0.082	0.143	0.119	0.082	0.510	0.510	0.510
	5	0.27	0.36	0.45	10.8	9.8	10.0	0.04	0.035	0.027	0.04	0.035	0.027	0.510	0.510	0.508
	6	0.28	0.36	0.54	18.0	12.9	18.6	0.083	0.156	0.138	0.083	0.156	0.138	0.510	0.506	0.506
	7	0.17	0.29	0.44	7.8	10.8	12.3	0.047	0.050	0.038	0.047	0.050	0.038	0.510	0.508	0.508
	8	0.27	0.29	0.43	9.0	2.2	5.1	0.010	0.007	0.015	0.010	0.007	0.015	0.508	0.508	0.508

Table 1.5; Permanent Deformation Parameters for Samples Made Using 60/70 Penetration Grade Bitumen and Tested at 60^0C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	4	0.13	0.13	0.19	6.5	6.3	7.0	0.027	0.025	0.023	0.027	0.025	0.023	0.511	0.510	0.510
	5	0.09	0.16	0.24	4.5	5.6	6.4	0.014	0.025	0.023	0.014	0.025	0.023	0.510	0.508	0.508
	6	0.16	0.21	0.24	6.7	6.7	13.3	0.050	0.023	0.075	0.050	0.023	0.075	0.508	0.506	0.508
	7	0.10	0.19	0.28	3.3	7.6	9.7	0.054	0.032	0.042	0.054	0.032	0.042	0.508	0.508	0.508
	8	0.11	0.18	0.22	5.5	11.0	13.3	0.018	0.038	0.072	0.018	0.038	0.072	0.510	0.508	0.508
2	4	0.19	0.28	0.39	5.7	10.9	15.4	0.018	0.036	0.154	0.018	0.036	0.154	0.511	0.510	0.508
	5	0.27	0.30	0.55	10.3	10.7	21.6	0.036	0.061	0.068	0.036	0.061	0.068	0.511	0.510	0.511
	6	0.18	0.22	0.25	7.1	10.9	13.3	0.020	0.088	0.059	0.020	0.088	0.059	0.510	0.508	0.510
	7	0.28	0.38	0.55	12.7	15.2	15.5	0.045	0.053	0.064	0.045	0.053	0.064	0.510	0.506	0.506
	8	0.13	0.35	0.44	3.1	13.9	17.9	0.010	0.100	0.111	0.010	0.100	0.111	0.508	0.508	0.508
3	4	0.25	0.31	0.39	10.3	11.9	13.8	0.032	0.040	0.046	0.032	0.040	0.046	0.510	0.510	0.510
	5	0.28	0.31	0.33	13.9	13.3	13.8	0.036	0.136	0.050	0.036	0.136	0.050	0.510	0.510	0.508
	6	0.26	0.29	0.37	7.6	9.2	16.2	0.033	0.031	0.065	0.033	0.031	0.065	0.508	0.508	0.508
	7	0.33	0.37	0.47	8.7	8.8	9.2	0.030	0.068	0.031	0.030	0.068	0.031	0.508	0.506	0.508
	8	0.19	0.28	0.40	4.1	7.6	11.5	0.014	0.033	0.072	0.014	0.033	0.072	0.506	0.506	0.508
4	4	0.19	0.20	0.25	11.7	12.0	15.4	0.089	0.056	0.039	0.089	0.056	0.039	0.510	0.510	0.510
	5	0.19	0.25	0.32	7.7	6.7	7.9	0.036	0.019	0.019	0.036	0.019	0.019	0.510	0.510	0.508
	6	0.12	0.16	0.24	7.7	5.3	10.0	0.055	0.037	0.054	0.055	0.037	0.054	0.510	0.506	0.506
	7	0.12	0.20	0.31	5.5	7.6	8.1	0.021	0.033	0.024	0.021	0.033	0.024	0.510	0.508	0.508
	8	0.19	0.20	0.30	6.6	2.3	3.5	0.028	0.010	0.012	0.028	0.010	0.012	0.508	0.508	0.508

Table 1.6; Permanent Deformation Parameters for Samples Made Using 80/100 Penetration Grade Bitumen and Tested at 20^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	4	1.94	2.52	3.49	46.2	53.3	59.3	0.308	0.429	0.189	0.308	0.429	0.189	0.506	0.506	0.506
	5	2.37	2.92	3.45	33.3	48.1	47.9	0.115	0.167	0.156	0.115	0.167	0.156	0.506	0.506	0.506
	6	1.89	2.48	3.21	36.9	53.3	75.4	0.267	0.25	0.25	0.267	0.25	0.25	0.506	0.506	0.506
	7	1.94	2.55	3.48	41.0	67.9	87.5	0.460	0.250	0.594	0.460	0.250	0.594	0.504	0.505	0.505
	8	1.17	1.48	1.76	34.7	51.9	53.1	0.500	0.750	0.400	0.500	0.750	0.400	0.506	0.505	0.505
2	4	1.19	1.74	2.29	61.3	69.7	76.9	0.200	0.215	0.558	0.200	0.215	0.558	0.506	0.506	0.506
	5	1.62	1.69	2.22	43.8	46.7	61.1	0.200	0.180	0.250	0.200	0.180	0.250	0.505	0.506	0.506
	6	2.10	2.19	2.95	23.1	16.7	23.3	0.115	0.083	0.100	0.115	0.083	0.100	0.506	0.506	0.506
	7	1.11	1.45	2.20	32.7	51.7	72.2	0.109	0.225	0.250	0.109	0.225	0.250	0.508	0.506	0.506
	8	0.74	1.35	1.99	10.8	30.8	69.2	0.062	0.277	0.615	0.062	0.277	0.615	0.505	0.506	0.506
3	4	1.56	1.92	2.41	56.2	52.6	52.0	0.243	0.192	0.160	0.243	0.192	0.160	0.510	0.510	0.508
	5	1.21	1.54	1.91	56.1	86.7	100.0	0.242	0.600	0.385	0.242	0.600	0.385	0.510	0.508	0.508
	6	1.63	2.11	2.79	66.7	89.7	103.0	0.229	0.308	0.500	0.229	0.308	0.500	0.510	0.508	0.508
	7	1.43	1.86	2.67	40.0	46.7	57.6	0.250	0.143	0.227	0.250	0.143	0.227	0.506	0.506	0.506
	8	1.22	2.03	2.67	34.8	44.0	54.0	0.178	0.180	0.190	0.178	0.180	0.190	0.506	0.506	0.506
4	4	1.53	1.89	2.49	46.2	53.3	57.6	0.160	0.143	0.182	0.160	0.143	0.182	0.506	0.506	0.506
	5	2.10	2.24	2.44	24.7	27.7	33.3	0.111	0.083	0.227	0.111	0.083	0.227	0.505	0.505	0.505
	6	1.56	2.16	2.79	30.5	54.7	100.0	0.100	0.231	0.273	0.100	0.231	0.273	0.504	0.506	0.505
	7	2.25	2.37	3.05	38.7	50.0	66.7	0.160	0.135	0.250	0.160	0.135	0.250	0.504	0.508	0.508
	8	1.53	2.66	3.44	44.8	52.4	68.8	0.171	0.500	0.594	0.171	0.500	0.594	0.506	0.506	0.506

Table 1.7; Permanent Deformation Parameters for Samples Made Using 80/100 Penetration Grade Bitumen and Tested at 30^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	4	1.05	1.36	1.89	25.3	28.3	32.8	0.120	0.100	0.112	0.120	0.100	0.112	0.506	0.506	0.506
	5	1.28	1.58	1.86	20.0	23.5	26.7	0.100	0.080	0.062	0.100	0.080	0.062	0.506	0.506	0.506
	6	1.02	1.34	1.74	19.0	25.0	40.0	0.133	0.075	0.133	0.133	0.075	0.133	0.506	0.506	0.506
	7	1.05	1.38	1.88	29.1	50.0	60.6	0.245	0.125	0.327	0.245	0.125	0.327	0.504	0.505	0.505
	8	0.63	0.80	0.95	20.0	28.1	30.0	0.300	0.400	0.154	0.300	0.400	0.154	0.506	0.505	0.505
2	4	0.89	1.31	1.72	46.7	53.3	57.4	0.138	0.160	0.400	0.138	0.160	0.400	0.506	0.506	0.506
	5	1.21	1.26	1.67	35.0	35.0	46.2	0.141	0.150	0.185	0.141	0.150	0.185	0.505	0.506	0.506
	6	1.58	1.64	2.22	17.8	12.3	14.5	0.100	0.025	0.036	0.100	0.025	0.036	0.506	0.506	0.506
	7	0.84	1.09	1.65	25.6	38.8	54.3	0.119	0.155	0.169	0.119	0.155	0.169	0.508	0.506	0.506
	8	0.55	1.01	1.49	8.5	24.2	46.7	0.055	0.127	0.125	0.055	0.127	0.125	0.505	0.506	0.506
3	4	1.08	1.32	1.67	37.6	35.1	44.1	0.114	0.100	0.138	0.114	0.100	0.138	0.510	0.510	0.508
	5	0.83	1.06	1.32	41.0	61.3	71.1	0.215	0.330	0.044	0.215	0.330	0.044	0.510	0.508	0.508
	6	1.13	1.46	1.92	46.7	63.3	71.1	0.160	0.250	0.272	0.160	0.250	0.272	0.510	0.508	0.508
	7	0.99	1.29	1.84	27.9	31.1	38.5	0.200	0.111	0.156	0.200	0.111	0.156	0.506	0.506	0.506
	8	0.84	1.40	1.84	26.7	30.0	38.4	0.092	0.125	0.128	0.092	0.125	0.128	0.506	0.506	0.506
4	4	0.95	1.17	1.55	28.7	32.7	34.9	0.092	0.105	0.112	0.092	0.105	0.112	0.506	0.506	0.506
	5	1.29	1.39	1.51	15.4	15.8	20.0	0.044	0.075	0.150	0.044	0.075	0.150	0.505	0.505	0.505
	6	0.96	1.34	1.73	20.0	33.3	60.9	0.064	0.175	0.188	0.064	0.175	0.188	0.504	0.506	0.505
	7	1.39	1.47	1.90	24.2	31.7	41.6	0.100	0.125	0.154	0.100	0.125	0.154	0.504	0.508	0.508
	8	0.95	1.65	2.13	27.7	34.9	42.3	0.120	0.200	0.154	0.120	0.200	0.154	0.506	0.506	0.506

Table 1.8; Permanent Deformation Parameters for Samples Made Using 80/100 Penetration Grade Bitumen and Tested at 40^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	4	0.58	0.75	1.04	13.3	15.8	5.1	0.071	0.063	0.055	0.071	0.063	0.055	0.506	0.506	0.506
	5	0.70	0.87	1.02	10.3	13.3	14.6	0.038	0.046	0.048	0.038	0.046	0.048	0.506	0.506	0.506
	6	0.56	0.74	0.96	10.6	13.3	20.5	0.064	0.050	0.077	0.064	0.050	0.077	0.506	0.506	0.506
	7	0.58	0.76	1.11	11.9	20.0	25.5	0.133	0.088	0.171	0.133	0.088	0.171	0.504	0.505	0.505
	8	0.35	0.44	0.52	10.2	15.0	17.0	0.053	0.069	0.090	0.053	0.069	0.090	0.506	0.505	0.505
2	4	0.49	0.72	0.95	26.7	29.6	31.8	0.082	0.084	0.200	0.082	0.084	0.200	0.506	0.506	0.506
	5	0.67	0.70	0.92	18.4	20.0	26.2	0.065	0.075	0.100	0.065	0.075	0.100	0.505	0.506	0.506
	6	0.87	0.90	1.22	8.5	6.4	10.4	0.032	0.032	0.044	0.032	0.032	0.044	0.506	0.506	0.506
	7	0.46	0.60	0.91	14.2	22.2	28.8	0.075	0.080	0.100	0.075	0.080	0.100	0.508	0.506	0.506
	8	0.30	0.56	0.82	5.2	13.9	28.7	0.033	0.073	0.277	0.033	0.073	0.277	0.505	0.506	0.506
3	4	0.59	0.73	0.92	21.7	20.4	26.1	0.094	0.060	0.080	0.094	0.060	0.080	0.510	0.510	0.508
	5	0.46	0.58	0.73	22.1	34.1	38.4	0.116	0.138	0.141	0.116	0.138	0.141	0.510	0.508	0.508
	6	0.62	0.80	1.06	24.9	34.9	40.0	0.089	0.154	0.164	0.089	0.154	0.164	0.510	0.508	0.508
	7	0.54	0.71	1.01	15.8	17.4	21.3	0.091	0.069	0.090	0.091	0.069	0.090	0.506	0.506	0.506
	8	0.46	0.77	1.01	15.8	17.1	21.0	0.048	0.067	0.086	0.048	0.067	0.086	0.506	0.506	0.506
4	4	0.57	0.70	0.93	18.2	19.8	20.5	0.055	0.063	0.062	0.055	0.063	0.062	0.506	0.506	0.506
	5	0.77	0.83	0.91	10.3	10.3	13.3	0.031	0.031	0.062	0.031	0.031	0.062	0.505	0.505	0.505
	6	0.59	0.81	1.04	11.9	20.5	38.2	0.044	0.092	0.109	0.044	0.092	0.109	0.504	0.506	0.505
	7	0.84	0.88	1.14	16.3	18.5	26.7	0.062	0.080	0.127	0.062	0.080	0.127	0.504	0.508	0.508
	8	0.57	0.99	1.28	16.7	21.3	26.0	0.060	0.080	0.320	0.060	0.080	0.320	0.506	0.506	0.506

Table 1.9; Permanent Deformation Parameters for Samples Made Using 80/100 Penetration Grade Bitumen and Tested at 50^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	4	0.44	0.57	0.79	11.1	11.9	13.3	0.075	0.036	0.046	0.075	0.036	0.046	0.506	0.506	0.506
	5	0.53	0.66	0.78	7.7	9.8	11.3	0.035	0.040	0.031	0.035	0.040	0.031	0.506	0.506	0.506
	6	0.43	0.56	0.73	8.2	10.8	15.8	0.038	0.040	0.048	0.038	0.040	0.048	0.506	0.506	0.506
	7	0.44	0.58	0.84	9.2	15.2	18.8	0.092	0.073	0.128	0.092	0.073	0.128	0.504	0.505	0.505
	8	0.27	0.33	0.40	7.6	11.6	12.7	0.045	0.053	0.112	0.045	0.053	0.112	0.506	0.505	0.505
2	4	0.34	0.50	0.66	17.5	18.6	22.5	0.056	0.071	0.150	0.056	0.071	0.150	0.506	0.506	0.506
	5	0.47	0.49	0.64	13.3	14.7	16.9	0.054	0.060	0.075	0.054	0.060	0.075	0.505	0.506	0.506
	6	0.61	0.63	0.85	0.100	4.6	6.9	0.031	0.013	0.032	0.031	0.013	0.032	0.506	0.506	0.506
	7	0.32	0.42	0.64	9.6	14.9	24.0	0.045	0.064	0.063	0.045	0.064	0.063	0.508	0.506	0.506
	8	0.21	0.39	0.58	6.7	9.2	19.8	0.042	0.067	0.152	0.042	0.067	0.152	0.505	0.506	0.506
3	4	0.37	0.46	0.58	13.8	12.3	15.0	0.057	0.038	0.050	0.057	0.038	0.050	0.510	0.510	0.508
	5	0.29	0.37	0.46	13.8	21.3	24.3	0.053	0.075	0.083	0.053	0.075	0.083	0.510	0.508	0.508
	6	0.39	0.50	0.67	16.3	21.2	25.0	0.050	0.076	0.113	0.050	0.076	0.113	0.510	0.508	0.508
	7	0.34	0.45	0.64	9.6	10.5	13.3	0.069	0.031	0.050	0.069	0.031	0.050	0.506	0.506	0.506
	8	0.29	0.48	0.64	8.3	10.3	13.3	0.038	0.029	0.044	0.038	0.029	0.044	0.506	0.506	0.506
4	4	0.36	0.44	0.58	10.4	11.8	13.3	0.038	0.042	0.044	0.038	0.042	0.044	0.506	0.506	0.506
	5	0.49	0.53	0.57	6.7	6.7	7.9	0.023	0.027	0.057	0.023	0.027	0.057	0.505	0.505	0.505
	6	0.36	0.51	0.65	7.8	13.3	23.3	0.025	0.067	0.075	0.025	0.067	0.075	0.504	0.506	0.505
	7	0.53	0.56	0.72	9.0	11.5	15.0	0.035	0.045	0.039	0.035	0.045	0.039	0.504	0.508	0.508
	8	0.36	0.62	0.81	10.0	12.9	16.2	0.044	0.097	0.092	0.044	0.097	0.092	0.506	0.506	0.506

Table 1.10; Permanent Deformation Parameters for Samples Made Using 80/100 Penetration Grade Bitumen and Tested at 60^0C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	4	0.27	0.35	0.48	6.7	7.5	8.5	0.029	0.031	0.027	0.029	0.031	0.027	0.506	0.506	0.506
	5	0.33	0.40	0.47	5.0	6.2	6.7	0.022	0.031	0.023	0.022	0.031	0.023	0.506	0.506	0.506
	6	0.26	0.34	0.44	5.1	7.1	10.3	0.031	0.031	0.031	0.031	0.031	0.031	0.506	0.506	0.506
	7	0.27	0.35	0.51	5.5	8.8	11.8	0.059	0.038	0.090	0.059	0.038	0.090	0.504	0.505	0.505
	8	0.16	0.20	0.24	4.8	7.1	8.0	0.034	0.090	0.036	0.034	0.090	0.036	0.506	0.505	0.505
2	4	0.28	0.40	0.53	14.3	15.9	18.2	0.043	0.054	0.118	0.043	0.054	0.118	0.506	0.506	0.506
	5	0.37	0.39	0.51	10.4	11.3	13.0	0.038	0.038	0.060	0.038	0.038	0.060	0.505	0.506	0.506
	6	0.49	0.51	0.68	6.0	3.6	5.8	0.020	0.018	0.025	0.020	0.018	0.025	0.506	0.506	0.506
	7	0.26	0.34	0.51	7.6	11.7	15.9	0.045	0.050	0.055	0.045	0.050	0.055	0.508	0.506	0.506
	8	0.17	0.31	0.46	2.7	7.9	15.1	0.015	0.039	0.077	0.015	0.039	0.077	0.505	0.506	0.506
3	4	0.31	0.38	0.48	10.8	10.3	12.8	0.044	0.030	0.038	0.044	0.030	0.038	0.510	0.510	0.508
	5	0.24	0.31	0.38	11.7	7.5	20.0	0.062	0.113	0.075	0.062	0.113	0.075	0.510	0.508	0.508
	6	0.32	0.42	0.55	13.3	17.6	20.0	0.055	0.064	0.082	0.055	0.064	0.082	0.510	0.508	0.508
	7	0.28	0.37	0.53	7.9	9.1	10.7	0.057	0.029	0.040	0.057	0.029	0.040	0.506	0.506	0.506
	8	0.24	0.40	0.53	7.3	10.5	10.5	0.027	0.071	0.038	0.027	0.071	0.038	0.506	0.506	0.506
4	4	0.25	0.31	0.41	7.6	8.3	8.7	0.048	0.025	0.031	0.048	0.025	0.031	0.506	0.506	0.506
	5	0.34	0.37	0.40	3.9	4.2	5.4	0.018	0.018	0.042	0.018	0.018	0.042	0.505	0.505	0.505
	6	0.26	0.36	0.46	5.2	8.7	16.2	0.018	0.048	0.046	0.018	0.048	0.046	0.504	0.506	0.505
	7	0.37	0.39	0.50	6.1	8.3	10.1	0.024	0.032	0.036	0.024	0.032	0.036	0.504	0.508	0.508
	8	0.25	0.44	0.56	7.3	9.3	11.4	0.026	0.038	0.050	0.026	0.038	0.050	0.506	0.506	0.506

Table 1.11; Permanent Deformation Parameters for Samples Made Using 180/200 Penetration Grade Bitumen and Tested at 20^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	4	1.48	1.92	2.67	35.0	41.0	46.0	0.175	0.154	0.125	0.175	0.154	0.125	0.506	0.506	0.506
	5	1.97	2.44	2.87	28.2	39.4	37.0	0.115	0.190	0.111	0.115	0.190	0.111	0.506	0.506	0.506
	6	1.87	2.46	3.18	37.8	51.5	70.4	0.250	0.227	0.222	0.250	0.227	0.222	0.506	0.506	0.506
	7	2.34	3.09	4.52	50.0	82.8	103.8	0.500	0.313	0.615	0.500	0.313	0.615	0.504	0.505	0.505
	8	1.93	2.43	2.91	56.4	81.8	91.7	0.711	1.192	0.639	0.711	1.192	0.639	0.506	0.505	0.505
2	4	0.91	1.33	1.75	47.2	53.3	60.0	0.154	0.150	0.367	0.154	0.150	0.367	0.506	0.506	0.506
	5	1.35	1.41	1.85	36.7	40.0	68.6	0.165	0.133	0.200	0.165	0.133	0.200	0.505	0.506	0.506
	6	2.08	2.17	2.93	22.7	15.4	18.5	0.080	0.077	0.056	0.080	0.077	0.056	0.506	0.506	0.506
	7	1.35	1.75	2.66	40.0	61.9	81.0	0.150	0.256	0.310	0.150	0.256	0.310	0.508	0.506	0.506
	8	1.21	2.23	3.29	18.3	53.8	107.4	0.182	0.308	0.389	0.182	0.308	0.389	0.505	0.506	0.506
3	4	1.20	1.47	1.84	41.7	41.7	50.1	0.125	0.125	0.138	0.125	0.125	0.138	0.510	0.510	0.508
	5	1.00	1.28	1.59	47.8	73.3	80.0	0.186	0.400	0.308	0.186	0.400	0.308	0.510	0.508	0.508
	6	1.62	2.09	2.76	63.6	91.4	100.0	0.215	0.370	0.386	0.215	0.370	0.386	0.510	0.508	0.508
	7	1.73	2.26	3.23	48.2	55.8	66.7	0.369	0.212	0.250	0.369	0.212	0.250	0.506	0.506	0.506
	8	2.01	3.35	4.40	59.7	66.7	87.2	0.250	0.250	0.308	0.250	0.250	0.308	0.506	0.506	0.506
4	4	1.17	1.45	1.90	33.3	38.3	41.0	0.125	0.133	0.123	0.125	0.133	0.123	0.506	0.506	0.506
	5	1.73	1.87	2.03	18.5	22.6	29.5	0.092	0.092	0.212	0.092	0.092	0.212	0.505	0.505	0.505
	6	1.54	2.14	2.76	32.8	52.0	95.2	0.092	0.320	0.310	0.092	0.320	0.310	0.504	0.506	0.505
	7	2.72	2.87	3.70	46.7	59.3	76.5	0.200	0.222	0.294	0.200	0.222	0.294	0.505	0.508	0.508
	8	2.53	4.40	5.67	73.0	88.9	113.6	0.262	0.500	0.545	0.262	0.500	0.545	0.506	0.506	0.506

Table 1.12; Permanent Deformation Parameters for Samples Made Using 180/200 Penetration Grade Bitumen and Tested at 30^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	4	0.80	1.04	1.44	20.0	21.8	26.7	0.120	0.091	0.100	0.120	0.091	0.100	0.506	0.506	0.506
	5	1.06	1.32	1.55	15.8	20.4	20.5	0.038	0.084	0.077	0.038	0.084	0.077	0.506	0.506	0.506
	6	1.01	1.33	1.72	21.3	26.7	42.7	0.160	0.111	0.160	0.160	0.111	0.160	0.506	0.506	0.506
	7	1.27	1.67	2.44	27.2	46.2	56.0	0.222	0.154	0.320	0.222	0.154	0.320	0.504	0.505	0.505
	8	1.04	1.32	1.57	31.5	45.0	50.0	0.382	0.656	0.286	0.382	0.656	0.286	0.506	0.505	0.505
2	4	0.68	1.00	1.32	35.0	40.0	45.9	0.113	0.120	0.289	0.113	0.120	0.289	0.506	0.506	0.506
	5	1.01	1.06	1.39	28.6	29.2	37.8	0.100	0.114	0.150	0.100	0.114	0.150	0.505	0.506	0.506
	6	1.56	1.63	2.19	14.4	14.4	14.5	0.062	0.046	0.040	0.062	0.046	0.040	0.506	0.506	0.506
	7	1.01	1.32	2.00	29.7	48.9	66.7	0.120	0.180	0.218	0.120	0.180	0.218	0.508	0.506	0.506
	8	0.91	1.67	2.46	14.4	42.2	80.0	0.085	0.233	0.511	0.085	0.233	0.511	0.505	0.506	0.506
3	4	0.83	1.01	1.27	29.7	26.7	34.1	0.112	0.100	0.111	0.112	0.100	0.111	0.510	0.510	0.508
	5	0.69	0.88	1.10	34.2	25.1	58.2	0.150	0.277	0.236	0.150	0.277	0.236	0.510	0.508	0.508
	6	1.12	1.44	1.90	46.7	61.7	73.8	0.220	0.200	0.369	0.220	0.200	0.369	0.510	0.508	0.508
	7	1.20	1.56	2.23	35.6	40.0	45.7	0.133	0.176	0.148	0.133	0.176	0.148	0.506	0.506	0.506
	8	1.38	2.31	3.04	40.0	48.0	60.0	0.175	0.171	0.200	0.175	0.171	0.200	0.506	0.506	0.506
4	4	0.73	0.90	1.18	21.7	24.4	26.7	0.075	0.077	0.089	0.075	0.077	0.089	0.506	0.506	0.506
	5	1.07	1.16	1.26	13.3	13.3	17.8	0.036	0.056	0.120	0.036	0.056	0.120	0.505	0.505	0.505
	6	0.96	1.33	1.71	20.0	35.1	60.8	0.064	0.140	0.200	0.064	0.140	0.200	0.504	0.506	0.505
	7	1.69	1.78	2.29	26.7	37.3	48.5	0.096	0.144	0.182	0.096	0.144	0.182	0.505	0.508	0.508
	8	1.57	2.73	3.52	44.4	56.1	71.4	0.200	0.267	0.286	0.200	0.267	0.286	0.506	0.506	0.506

Table 1.13; Permanent Deformation Parameters for Samples Made Using 180/200 Penetration Grade Bitumen and Tested at 40^0C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	4	0.44	0.57	0.79	10.3	12.1	13.3	0.046	0.045	0.046	0.046	0.045	0.046	0.506	0.506	0.506
	5	0.59	0.72	0.85	9.1	11.7	13.3	0.050	0.063	0.046	0.050	0.063	0.046	0.506	0.506	0.506
	6	0.56	0.73	0.95	10.9	15.0	23.6	0.073	0.063	0.092	0.073	0.063	0.092	0.506	0.506	0.506
	7	0.70	0.92	1.34	15.0	26.2	32.1	0.163	0.077	0.200	0.163	0.077	0.200	0.504	0.505	0.505
	8	0.57	0.72	0.86	17.8	25.0	28.5	0.078	0.106	0.115	0.078	0.106	0.115	0.506	0.505	0.505
2	4	0.38	0.55	0.72	20.0	22.1	25.8	0.056	0.059	0.153	0.056	0.059	0.153	0.506	0.506	0.506
	5	0.56	0.58	0.76	15.5	17.0	21.2	0.055	0.056	0.082	0.055	0.056	0.082	0.505	0.506	0.506
	6	0.86	0.89	1.21	9.2	6.9	0.156	0.031	0.030	0.044	0.031	0.030	0.044	0.506	0.506	0.506
	7	0.56	0.72	1.10	16.7	26.7	30.9	0.070	0.094	0.109	0.070	0.094	0.109	0.508	0.506	0.506
	8	0.50	0.92	1.36	8.2	22.6	45.0	0.050	0.115	0.200	0.050	0.115	0.200	0.505	0.506	0.506
3	4	0.48	0.58	0.73	16.7	17.0	20.0	0.055	0.056	0.063	0.055	0.056	0.063	0.510	0.510	0.508
	5	0.40	0.51	0.63	19.0	29.1	34.6	0.088	0.155	0.150	0.088	0.155	0.150	0.510	0.508	0.508
	6	0.64	0.83	1.10	26.7	36.4	40.0	0.088	0.154	0.182	0.088	0.154	0.182	0.510	0.508	0.508
	7	0.69	0.90	1.28	20.0	22.6	26.7	0.147	0.125	0.067	0.147	0.125	0.067	0.506	0.506	0.506
	8	0.80	1.33	1.75	23.6	26.7	35.9	0.092	0.100	0.123	0.092	0.100	0.123	0.506	0.506	0.506
4	4	0.44	0.54	0.71	13.3	14.0	16.6	0.046	0.050	0.050	0.046	0.050	0.050	0.506	0.506	0.506
	5	0.64	0.69	0.75	7.5	8.3	5.0	0.025	0.026	0.075	0.025	0.026	0.075	0.505	0.505	0.505
	6	0.57	0.80	1.03	12.2	20.8	36.4	0.042	0.096	0.100	0.042	0.096	0.100	0.504	0.506	0.505
	7	1.01	1.07	1.38	17.1	23.0	16.3	0.067	0.073	0.111	0.067	0.073	0.111	0.505	0.508	0.508
	8	0.94	1.64	2.11	27.8	32.8	42.7	0.100	0.185	0.218	0.100	0.185	0.218	0.506	0.506	0.506

Table 1.14; Permanent Deformation Parameters for Samples Made Using 180/200 Penetration Grade Bitumen and Tested at 50^0C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	4	0.34	0.43	0.60	7.6	9.2	11.0	0.032	0.038	0.034	0.032	0.038	0.034	0.506	0.506	0.506
	5	0.45	0.55	0.65	6.7	8.3	9.2	0.029	0.035	0.031	0.029	0.035	0.031	0.506	0.506	0.506
	6	0.42	0.55	0.72	8.6	10.9	16.4	0.067	0.048	0.046	0.067	0.048	0.046	0.506	0.506	0.506
	7	0.53	0.70	1.02	5.9	20.0	23.3	0.118	0.067	0.150	0.118	0.067	0.150	0.504	0.505	0.505
	8	0.44	0.55	0.66	13.3	19.3	21.6	0.059	0.219	0.088	0.059	0.219	0.088	0.506	0.505	0.505
2	4	0.26	0.39	0.51	13.8	15.5	17.3	0.041	0.040	0.114	0.041	0.040	0.114	0.506	0.506	0.506
	5	0.39	0.41	0.53	10.3	11.3	15.0	0.040	0.050	0.055	0.040	0.050	0.055	0.505	0.506	0.506
	6	0.60	0.63	0.85	5.8	4.0	6.4	0.026	0.012	0.018	0.026	0.012	0.018	0.506	0.506	0.506
	7	0.39	0.51	0.77	11.1	17.8	23.1	0.040	0.071	0.096	0.040	0.071	0.096	0.508	0.506	0.506
	8	0.35	0.64	0.95	0.079	15.8	32.0	0.018	0.100	0.312	0.018	0.100	0.312	0.505	0.506	0.506
3	4	0.29	0.35	0.44	10.0	9.6	12.2	0.045	0.025	0.036	0.045	0.025	0.036	0.510	0.510	0.508
	5	0.24	0.31	0.38	11.8	17.1	20.0	0.064	0.106	0.075	0.064	0.106	0.075	0.510	0.508	0.508
	6	0.39	0.50	0.66	16.0	20.5	24.0	0.065	0.062	0.087	0.065	0.062	0.087	0.510	0.508	0.508
	7	0.41	0.54	0.77	11.8	13.3	16.4	0.131	0.050	0.062	0.131	0.050	0.062	0.506	0.506	0.506
	8	0.48	0.80	1.05	14.3	16.4	21.0	0.050	0.062	0.080	0.050	0.062	0.080	0.506	0.506	0.506
4	4	0.27	0.34	0.45	8.3	9.3	9.7	0.029	0.033	0.035	0.029	0.033	0.035	0.506	0.506	0.506
	5	0.41	0.44	0.48	5.1	5.3	6.7	0.015	0.020	0.050	0.015	0.020	0.050	0.505	0.505	0.505
	6	0.36	0.50	0.65	7.5	12.7	22.4	0.025	0.06	0.073	0.025	0.06	0.073	0.504	0.506	0.505
	7	0.64	0.67	0.87	11.0	13.3	18.5	0.027	0.050	0.08	0.027	0.050	0.08	0.505	0.508	0.508
	8	0.59	1.03	1.33	17.5	22.0	27.4	0.063	0.100	0.147	0.063	0.100	0.147	0.506	0.506	0.506

Table 1.15; Permanent Deformation Parameters for Samples Made Using 180/200 Penetration Grade Bitumen and Tested at 60^0C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	4	0.20	0.27	0.37	4.9	5.5	6.7	0.038	0.022	0.020	0.038	0.022	0.020	0.506	0.506	0.506
	5	0.27	0.34	0.40	4.1	5.4	6.7	0.019	0.031	0.027	0.019	0.031	0.027	0.506	0.506	0.506
	6	0.26	0.34	0.44	5.2	7.1	10.3	0.034	0.025	0.031	0.034	0.025	0.031	0.506	0.506	0.506
	7	0.32	0.43	0.62	6.8	11.5	14.4	0.073	0.038	0.100	0.073	0.038	0.100	0.504	0.505	0.505
	8	0.27	0.34	0.40	6.7	11.6	12.7	0.119	0.175	0.062	0.119	0.175	0.062	0.506	0.505	0.505
2	4	0.21	0.31	0.41	11.2	12.3	13.8	0.033	0.039	0.085	0.033	0.039	0.085	0.506	0.506	0.506
	5	0.31	0.33	0.43	8.3	9.2	11.7	0.031	0.038	0.046	0.031	0.038	0.046	0.505	0.506	0.506
	6	0.48	0.50	0.68	5.1	4.0	5.5	0.015	0.015	0.024	0.015	0.015	0.024	0.506	0.506	0.506
	7	0.31	0.41	0.62	9.2	14.6	19.2	0.044	0.054	0.063	0.044	0.054	0.063	0.508	0.506	0.506
	8	0.28	0.51	0.76	4.2	12.4	24.2	0.021	0.067	0.125	0.021	0.067	0.125	0.505	0.506	0.506
3	4	0.24	0.29	0.37	8.3	7.9	9.8	0.029	0.125	0.027	0.029	0.125	0.027	0.510	0.510	0.508
	5	0.20	0.25	0.32	9.7	14.5	16.3	0.040	0.086	0.063	0.040	0.086	0.063	0.510	0.508	0.508
	6	0.32	0.41	0.55	8.8	17.9	20.0	0.050	0.069	0.100	0.050	0.069	0.100	0.510	0.508	0.508
	7	0.34	0.45	0.64	10.0	11.1	13.3	0.063	0.038	0.050	0.063	0.038	0.050	0.506	0.506	0.506
	8	0.40	0.66	0.87	11.8	13.9	17.4	0.044	0.065	0.074	0.044	0.065	0.074	0.506	0.506	0.506
4	4	0.19	0.24	0.31	5.6	6.7	7.1	0.018	0.018	0.025	0.018	0.018	0.025	0.506	0.506	0.506
	5	0.28	0.31	0.33	3.3	3.3	4.8	0.014	0.013	0.038	0.014	0.013	0.038	0.505	0.505	0.505
	6	0.25	0.35	0.45	5.4	8.8	15.8	0.020	0.034	0.050	0.020	0.034	0.050	0.504	0.506	0.505
	7	0.45	0.47	0.61	7.8	9.7	13.3	0.023	0.038	0.059	0.023	0.038	0.059	0.505	0.508	0.508
	8	0.42	0.72	0.93	12.3	15.1	19.2	0.046	0.063	0.112	0.046	0.063	0.112	0.506	0.506	0.506

Table 2.1; Permanent Deformation Parameters for Samples Made Using Optimum Bitumen Content of 60/70 Penetration Grade Bitumen and Tested at 20^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Optimum bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	5.1	0.628	1.215	1.741	29.14	39.86	46.91	0.122	0.273	0.165	0.122	0.273	0.165	0.510	0.508	0.508
2	5.3	0.900	1.189	1.989	43.35	48.82	83.51	0.119	0.220	0.265	0.119	0.220	0.265	0.511	0.509	0.511
3	5.5	1.345	1.505	1.765	45.90	66.70	75.90	0.172	0.372	0.323	0.172	0.372	0.323	0.509	0.509	0.508
4	5.8	1.230	1.592	2.34	72.50	54.84	78.6	0.348	0.417	0.485	0.348	0.417	0.485	0.510	0.507	0.506

Table 2.2; Permanent Deformation Parameters for Samples Made Using Optimum Bitumen Content of 60/70 Penetration Grade Bitumen and Tested at 30^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Optimum bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	5.1	0.344	0.659	0.941	17.59	20.67	24.93	0.099	0.117	0.070	0.099	0.117	0.070	0.510	0.508	0.508
2	5.3	0.790	0.896	1.492	27.65	36.05	62.96	0.100	0.172	0.249	0.100	0.172	0.249	0.511	0.509	0.511
3	5.5	0.930	1.040	1.215	32.10	36.90	53.45	0.119	0.200	0.171	0.119	0.200	0.171	0.509	0.509	0.508
4	5.8	0.766	0.988	1.448	44.22	31.56	32.50	0.214	0.117	0.171	0.214	0.117	0.171	0.510	0.507	0.506

Table 2.3; Permanent Deformation Parameters for Samples Made Using Optimum Bitumen Content of 60/70 Penetration Grade Bitumen and Tested at 40^0C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Optimum bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	5.1	0.192	0.360	0.520	8.51	11.57	14.10	0.024	0.063	0.053	0.024	0.063	0.053	0.510	0.508	0.508
2	5.3	0.432	0.495	0.821	18.17	20.33	33.03	0.057	0.114	0.133	0.057	0.114	0.133	0.511	0.509	0.511
3	5.5	0.510	0.570	0.670	16.65	20.65	29.05	0.058	0.088	0.141	0.058	0.088	0.141	0.509	0.509	0.508
4	5.8	0.462	0.594	0.872	26.64	19.88	27.16	0.134	0.100	0.136	0.134	0.100	0.136	0.510	0.507	0.506

Table 2.4; Permanent Deformation Parameters for Samples Made Using Optimum Bitumen Content of 60/70 Penetration Grade Bitumen and Tested at 50^0C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Optimum bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	5.1	0.142	0.278	0.391	6.67	11.43	11.72	0.044	0.058	0.055	0.044	0.058	0.055	0.510	0.508	0.508
2	5.3	0.307	0.347	0.572	12.16	14.10	24.14	0.038	0.052	0.081	0.038	0.052	0.081	0.511	0.509	0.511
3	5.5	0.32	0.36	0.415	10.75	13.20	18.50	0.043	0.052	0.073	0.043	0.052	0.073	0.509	0.509	0.508
4	5.8	0.278	0.360	0.522	16.56	12.28	16.88	0.074	0.132	0.116	0.074	0.132	0.116	0.510	0.507	0.506

Table 2.5; Permanent Deformation Parameters for Samples Made Using Optimum Bitumen Content of 60/70 Penetration Grade Bitumen and Tested at 60^0C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Optimum bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	5.1	0.097	0.165	0.240	4.72	5.71	7.09	0.023	0.025	0.025	0.023	0.025	0.025	0.510	0.508	0.508
2	5.3	0.243	0.276	0.460	9.34	10.76	19.11	0.031	0.069	0.065	0.031	0.069	0.065	0.511	0.509	0.511
3	5.5	0.270	0.300	0.350	10.75	11.25	15.0	0.035	0.084	0.058	0.035	0.084	0.058	0.509	0.509	0.508
4	5.8	0.134	0.178	0.256	7.70	5.58	9.58	0.051	0.033	0.047	0.051	0.033	0.047	0.510	0.507	0.506

Table 2.6; Permanent Deformation Parameters for Samples Made Using Optimum Bitumen Content of 80/100 Penetration Grade Bitumen and Tested at 20^0C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Optimum bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	5.2	2.274	2.832	3.402	34.02	49.14	53.4	0.145	0.184	0.175	0.145	0.184	0.175	0.506	0.506	0.506
2	5.5	1.860	1.940	2.585	33.45	31.70	42.20	0.158	0.132	0.175	0.158	0.132	0.175	0.506	0.506	0.506
3	6.0	1.630	2.110	2.790	66.70	89.70	103.0	0.229	0.308	0.500	0.229	0.308	0.500	0.510	0.508	0.508
4	5.9	1.614	2.168	2.755	29.92	52.00	93.33	0.101	0.216	0.268	0.101	0.216	0.268	0.504	0.505	0.505

Table 2.7; Permanent Deformation Parameters for Samples Made Using Optimum Bitumen Content of 80/100 Penetration Grade Bitumen and Tested at 30^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Optimum bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	5.2	1.228	1.532	1.836	19.80	23.80	29.36	0.107	0.079	0.076	0.107	0.079	0.076	0.506	0.506	0.506
2	5.5	1.395	1.450	1.945	26.40	23.65	30.35	0.121	0.088	0.111	0.121	0.088	0.111	0.506	0.506	0.506
3	6.0	1.130	1.460	1.920	46.70	63.30	71.10	0.160	0.250	0.272	0.160	0.250	0.272	0.510	0.508	0.508
4	5.9	0.993	1.345	1.708	19.54	31.55	56.81	0.062	0.165	0.184	0.062	0.165	0.184	0.504	0.505	0.505

Table 2.8; Permanent Deformation Parameters for Samples Made Using Optimum Bitumen Content of 80/100 Penetration Grade Bitumen and Tested at 40^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Optimum bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	5.2	0.672	0.844	1.008	10.36	13.30	15.78	0.043	0.047	0.054	0.043	0.047	0.054	0.506	0.506	0.506
2	5.5	0.770	0.800	1.070	13.45	13.20	18.30	0.061	0.054	0.072	0.061	0.054	0.072	0.506	0.506	0.506
3	6.0	0.620	0.800	1.060	24.90	34.90	40.00	0.089	0.154	0.164	0.089	0.154	0.164	0.510	0.508	0.508
4	5.9	0.608	0.812	1.027	11.74	19.48	35.71	0.043	0.086	0.104	0.043	0.086	0.104	0.504	0.505	0.505

Table 2.9; Permanent Deformation Parameters for Samples Made Using Optimum Bitumen Content of 80/100 Penetration Grade Bitumen and Tested at 50^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Optimum bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	5.2	0.510	0.640	0.770	7.80	10.00	12.20	0.036	0.040	0.034	0.036	0.040	0.034	0.506	0.506	0.506
2	5.5	0.540	0.560	0.745	7.40	9.65	11.90	0.043	0.037	0.054	0.043	0.037	0.054	0.506	0.506	0.506
3	6.0	0.390	0.500	0.670	16.30	21.20	25.00	0.050	0.076	0.113	0.050	0.076	0.113	0.510	0.508	0.508
4	5.9	0.373	0.512	0.642	7.69	12.64	21.76	0.025	0.063	0.073	0.025	0.063	0.073	0.504	0.505	0.505

Table 2.10; Permanent Deformation Parameters for Samples Made Using Optimum Bitumen Content of 80/100 Penetration Grade Bitumen and Tested at 60^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Optimum bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	5.2	0.316	0.388	0.464	5.02	6.38	7.42	0.024	0.031	0.025	0.024	0.031	0.025	0.506	0.506	0.506
2	5.5	0.400	0.450	0.595	8.20	7.45	9.40	0.029	0.028	0.043	0.029	0.028	0.043	0.506	0.506	0.506
3	6.0	0.320	0.420	0.550	13.30	17.60	20.00	0.055	0.064	0.082	0.055	0.064	0.082	0.510	0.508	0.508
4	5.9	0.268	0.361	0.454	5.07	8.25	15.12	0.018	0.045	0.046	0.018	0.045	0.046	0.504	0.505	0.505

Table 2.11; Permanent Deformation Parameters for Samples Made Using Optimum Bitumen Content of 180/200 Penetration Grade Bitumen and Tested at 20^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Optimum bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	5.5	1.920	2.450	3.025	33.00	45.45	53.70	0.183	0.209	0.167	0.183	0.209	0.167	0.506	0.506	0.506
2	6.1	2.080	2.170	2.930	22.70	15.40	18.50	0.080	0.077	0.056	0.080	0.077	0.056	0.506	0.506	0.506
3	6.1	1.620	2.090	2.760	63.60	91.40	100.00	0.215	0.370	0.386	0.215	0.370	0.386	0.510	0.508	0.508
4	6.2	1.776	2.286	2.948	35.58	53.46	91.46	0.114	0.300	0.307	0.114	0.300	0.307	0.504	0.506	0.505

Table 2.12; Permanent Deformation Parameters for Samples Made Using Optimum Bitumen Content of 180/200 Penetration Grade Bitumen and Tested at 30^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Optimum bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	5.5	1.035	1.325	1.635	18.55	23.55	31.60	0.099	0.098	0.119	0.099	0.098	0.119	0.506	0.506	0.506
2	6.1	1.560	1.630	2.190	14.40	14.40	14.50	0.062	0.046	0.040	0.062	0.046	0.040	0.506	0.506	0.506
3	6.1	1.120	1.440	1.900	46.70	61.70	73.80	0.220	0.200	0.369	0.220	0.200	0.369	0.510	0.508	0.508
4	6.2	1.106	1.420	1.826	21.34	35.54	58.34	0.070	0.141	0.196	0.070	0.141	0.196	0.504	0.506	0.505

Table 2.13; Permanent Deformation Parameters for Samples Made Using Optimum Bitumen Content of 180/200 Penetration Grade Bitumen and Tested at 40^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Optimum bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	5.5	0.575	0.725	0.900	10.00	13.35	18.45	0.062	0.063	0.069	0.062	0.063	0.069	0.506	0.506	0.506
2	6.1	0.860	0.890	1.210	9.20	6.90	10.40	0.031	0.030	0.044	0.031	0.030	0.044	0.506	0.506	0.506
3	6.1	0.640	0.830	1.100	26.70	36.40	40.0	0.088	0.154	0.182	0.088	0.154	0.182	0.510	0.508	0.508
4	6.2	0.658	0.854	1.205	14.65	21.24	32.38	0.047	0.091	0.102	0.047	0.091	0.102	0.504	0.506	0.505

Table 2.14; Permanent Deformation Parameters for Samples Made Using Optimum Bitumen Content of 180/200 Penetration Grade Bitumen and Tested at 50^{0}C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Optimum bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	5.5	0.435	0.550	0.685	7.65	9.60	12.80	0.048	0.042	0.039	0.048	0.042	0.039	0.506	0.506	0.506
2	6.1	0.600	0.630	0.850	5.80	4.00	6.40	0.026	0.012	0.018	0.026	0.012	0.018	0.506	0.506	0.506
3	6.1	0.390	0.500	0.660	16.00	20.50	24.00	0.065	0.062	0.087	0.065	0.062	0.087	0.510	0.508	0.508
4	6.2	0.416	0.534	0.694	8.20	12.82	23.18	0.025	0.058	0.074	0.025	0.058	0.074	0.504	0.506	0.505

Table 2.15; Permanent Deformation Parameters for Samples Made Using Optimum Bitumen Content of 180/200 Penetration Grade Bitumen and Tested at 60^0C

Blend		Max Stress [Mpa]			Tangent Stiffness [Mpa]			Stress at initiation of plasticity [Mpa]			Stress at initiation of Dilation [Mpa]			Poisson's Ratio		
	Optimum bitumen content [%]	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa	0.1Mpa	0.2Mpa	0.4Mpa
1	5.5	0.265	0.340	0.420	4.65	6.25	8.50	0.027	0.028	0.029	0.027	0.028	0.029	0.506	0.506	0.506
2	6.1	0.480	0.500	0.680	5.10	4.00	5.50	0.015	0.015	0.024	0.015	0.015	0.024	0.506	0.506	0.506
3	6.1	0.320	0.410	0.550	8.80	17.90	20.00	0.050	0.069	0.100	0.050	0.069	0.100	0.510	0.508	0.508
4	6.2	0.290	0.374	0.482	5.88	8.98	15.30	0.021	0.035	0.052	0.021	0.035	0.052	0.504	0.506	0.505

Note; See the other part of appendix 2 in the attached CD-ROM

a) The raw triaxial compression test data as recorded in the laboratory

b) The Vertical stress versus the axial strain curves